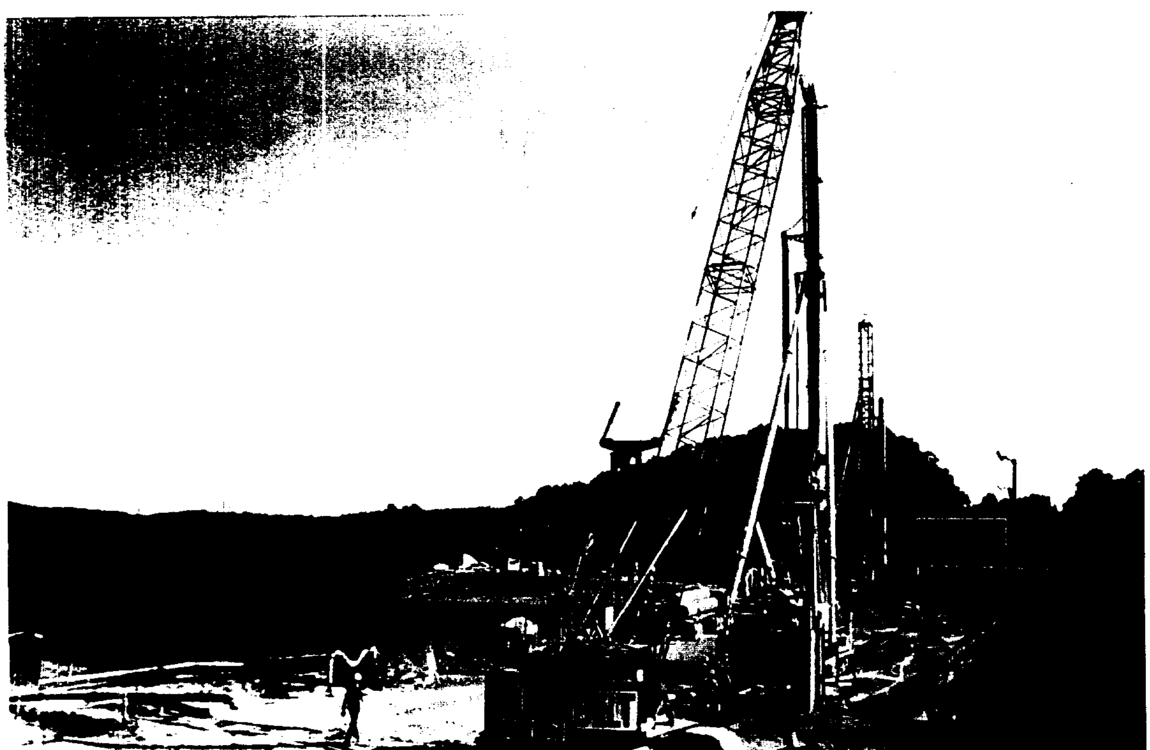
Beaver Dam - White River, Arkansas

U.S. Army Corps
of Engineers
Little Rock District

Beaver Dam Secant Pile Wall Completion Report



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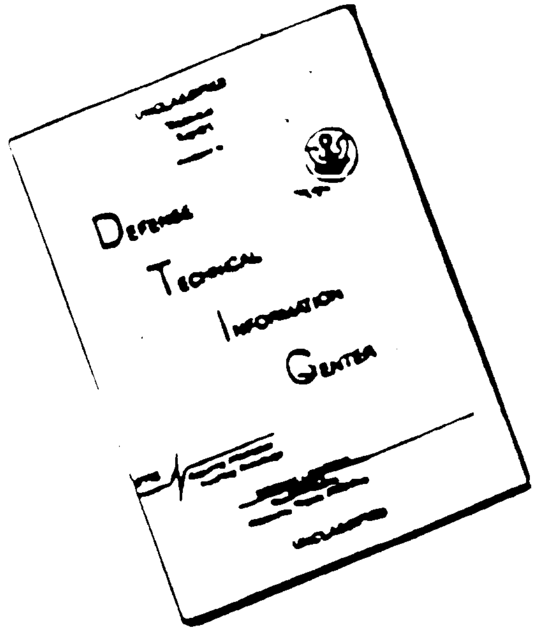

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13. ABSTRACT

The report presents a comprehensive project history, beginning in 1966 with the initial Beaver Lake (located near Eureka Springs, in northwest Arkansas, U.S.A.) impoundment against Dike 1, which produced observable surface seepage from five areas downstream of Dike 1. Seepage continued to increase due to limestone rock weathering and solutioning, combined with removal of colloidal sized soil material from the overburden and weathered rock zones, particularly along two faults that formed a graben beneath Dike 1. The pervasive seepage distress evolved until a muddy water boil was discovered during a "pool of record" event in December 1984, and ultimately warranted a remedial construction project. The selected method for alleviating the seepage distress consisted of a 1,475-foot-long concrete diaphragm wall located immediately upstream of Dike 1 and penetrating to depths of 80 to 185 feet. Utilizing a request for proposals/technical evaluation/price negotiation contractual process, the construction was accomplished by two separate joint venture contractors under two contracts. The first contractor performed site preparation and constructed an upstream work platform; however, the contractor failed to complete the project because the specialized rock mill excavation equipment could not excavate the proposed overlapping trench segments into the high compressive strength unweathered rock that typically exceeded 15,000 psi. The inability to construct the wall resulted in the first contract being terminated for convenience to the Government. The second contractor, after overcoming shaft excavation instability problems, completed the wall using a large diameter down-the-hole air percussion hammer drill for excavation. A total of 739 overlapping, 34-inch diameter shafts were drilled on 24-inch centers. The shafts were filled with concrete using the gravity tremie method; thus, creating a continuous concrete secant pile wall with a minimum width of 18 inches. The completed wall successfully eliminated all observable seepage downstream of the dike, and significantly reduced piezometric levels. The project construction was completed in December 1994, at an approximate cost of $33,535,000.

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9. Concrete Secant Piles						
10. Down Hole Air Percussion Hammer Drill						
11. Work Platform						
12. Excavation						
13. Stabilization						
14. Verticality						
15. Concrete Quality						
16. Aggregate Segregation						
17. Pile Joint						
18. Grouting						
19. Seepage Elimination						

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Beaver Dam Secant Pile Wall Construction Completion Report

SECTION 1

1. INTRODUCTION.

1.1 Damsite Location. Beaver Dam is located on the White River, 609 miles above its mouth, in Carroll County, Northwest Arkansas. Its proximity location is 6 miles northwest of Eureka Springs, Arkansas on State Highway 187 (Plate A-1). The dam, constructed between November 1960 and June 1966, is a concrete gravity and compacted zoned earth embankment structure with a 1,332 foot concrete section, a 1,242-foot earth section, and three auxiliary saddle dikes. The main dam has a maximum height of 228 feet above the stream bed. The dam spillway is located in the concrete gravity section and contains 7 bays, each 40 feet wide. The project also produces hydroelectric power with two 56,000 kilowatt capacity power units. The top elevation of the flood control pool is 1,130 N.G.V.D. The maximum pool elevation is 1,137 N.G.V.D. with a top of dam elevation of 1142. Due to concerns for seepage distress, the maximum flood control pool was temporarily lowered to Elevation 1128, from January 1985 to January 1995.

1.2 Location of Structure. The construction site, located at Dike 1, is directly adjacent to the north end of the Beaver Dam main embankment on Arkansas Highway 187 (Plates A-2 & A-3). The construction project consisted of constructing a remedial concrete seepage cutoff wall (Plate A-4) varying from 80 to 185 feet deep and extending from dam station 62+00 to 76+75 (1,475 feet). The cutoff wall was offset 65 feet upstream from the dike center line axis which required the construction of a 75 foot wide work platform (Plate A-4). Most of the work platform construction began at lake level depth with the finished surface being at elevation 1130.

1.3 Construction Authority. Beaver Dam Reservoir was authorized for flood control, power, and other purposes by the Flood Control Act of 1954, approved 3 September 1954 (Public Law 780, 83d Congress, 2d session) as recommended by the Chief of Engineers in House Document No. 499, 83d Congress, 2d session. The inclusion of municipal and industrial water supply as a project purpose was authorized by the Water Supply Act of 1958 (Public Law 85-500, 85th Congress). The first contractor to attempt the Dike 1 cutoff wall project (Solentanche/Rodio/Nicholson Joint Venture) operated under Contract No. DACWD03-89-C-0041 and the latter contractor (Rodio-Nicholson Joint Venture - RNJV) under Contract No. DACW03-92-C-0039.

1.4 Purpose of Report. This report is presented to give a general discussion of the second construction contract that successfully completed the cutoff wall. It contains background and foundation information, details of the contract, general description of construction procedures, and discusses problems

encountered in the Rodio-Nicholson contract. Since the first contract with Soletanche-Rodio-Nicholson was terminated without completing any portion of the wall, only brief and general discussions are included regarding their effort. However their effort and resulting problems can be found in litigation documents produced for the settlement of outstanding claims that Soletanche-Rodio-Nicholson filed against the Corps. These documents can be referenced in the Little Rock District Corps of Engineers, Office of Council, located in Little Rock, Arkansas.

1.5 Project History. The site selection for Beaver Dam and auxiliary dikes was made in 1957. At that time a graben was known to exist under Dike 1 and was recognized as a problem area (Plate B-4). Foundation treatment consisted of a grout curtain installed at the time of construction. Dike 1 (completed in 1966) was originally designed as a multi-zoned structure with the more impervious material being placed in the core (Plate A-5). However, due to the abundance of impervious material near the site, the structure was actually constructed to basically a single-zoned impervious embankment. Random fill used in construction consisted of low-plasticity red clay with angular gravel (Residual Boone Limestone).

Shortly after construction and during the initial reservoir filling (1966) against Dike 1, seepage was detected from 5 areas (Plate A-6) downstream of the dike, totaling approximately 800 gpm at normal lake level. To monitor the seepage flows, a temporary weir was installed below seepage Exits 1, 2, and 3 in 1966. A permanent Parshall Flume, referred to as Flume No. 1, was installed in 1967 to intercept all flow from known seepage exits. Twenty six piezometers were installed between 1968 and 1972 to monitor seepage conditions in overburden, weathered rock, and sound rock. Remedial grouting, performed intermittently between 1968 and 1971, reduced observable flows to about 500 gpm at normal lake level. Plate A-12 shows the approximate depth of the grout curtain. It was pointed out in the geological analysis of the Dike 1 Remedial Foundation Treatment Report (1972) that due to the characteristics of the Boone Formation, i.e., numerous open and clay filled cavities, channels, porous (sponge like) rock, with deep, intensely weathered, pervious zones in the foundation and fault areas, that grouting would not solve the seepage problem.

By early 1975, a seep at Exit 6 and a small wet area at Exit 7 were observed (Plate A-6). Seepage at Exit 6 continued to grow until by 1982 a major seepage exit had developed (10 Gal/Min flow). Seepage in Exit 7 remained the same until the pool of record in December 1984 (el. 1130.4). Seepage Exit 9 was also discovered about that time and was found to have muddy flows. Exit 9 lies almost 500 feet downstream of the Dike 1 toe. On 2 January 1985 (pool el. 1125.1) seepage was observed jetting approximately 1.25 inches vertically and producing about 6 gpm at

Exit 7. Although flowing clear water and less volume than seepage at Exit 9, the Exit 7 seep was significant because it was located at the Dike 1/left abutment groin at about one third of the dike height, thus being only about two hundred feet from the lake source. Other minor seepage areas (Exits 7A and 10) were also first observed after the December Pool of Record.

A seepage investigation was initiated in Feb. 1985 and continued for several years. A French Drain was constructed in 1985 down gradient of seepage Exit 6 to collect, measure and discharge the seepage flow. A second Parshall Flume was installed in 1985 downstream from the first flume, to monitor seepage from Exit 6 that was bypassing Flume No. 1. Thirty new piezometers were installed in 1985 at Dike 1 which brought the total number to 56 (Plates A-6 & C-1). An automated recording system was installed in 1986 for all piezometers on Dike 1 as well as 16 piezometers on the Beaver Dam main embankment and 15 piezometers on Dike 3.

In February 1988 a decision was made to proceed with the construction of a concrete cutoff wall. The contract was awarded to Soletanche-Rodio-Nicholson on 14 June 1989. Construction began on 16 February 1990 with site preparation. A 75 foot wide work platform was constructed on the upstream slope of Dike 1 from stations 61+70 to 77+06 at el. 1130 (Plates A-4 & A-5). Generally, the work platform was made of uncompacted fill (below the water surface), semicompacted (for two feet above the water surface) and compacted (for 1 foot above the semicompacted) granular material. On top of the compacted granular material was 1 to 5 feet of random fill (generally impervious lean clay). Specifications for random fill could also be met using the granular fill which was used as needed for that purpose. Excavation for the cutoff wall was begun on 12 July 1990 using the contractors model 12000 Hydrofraise (rockmill). Four panels were partially excavated (Plate A-7) but it quickly became apparent that the equipment was incapable of economically excavating the hard unweathered rock types at the site. Several different cutter head configurations were used without success. Excavation ceased on 1 August 1990 and the contractor eventually claimed differing site conditions after offering several alternative proposals, none of which were acceptable to the Government. However, it should be noted that prior to SRNJV submitting their best and final bid to the government, the contractor had in his proposal that a down the hole hammer (Super Drill) would serve as a backup to the hydrofraise in the harder rock. This was also the understanding of the Engineering Technical Evaluation Team that reviewed the proposal. At the time of the best and final bid, the contractor was allowed (without the knowledge of the Technical Evaluation Team) to withdraw the Super Drill from their initial bid in order to lower his bid and enhance his chance of getting the job.

A Resolicitation Request for Proposals (RFP) to construct the concrete cutoff wall was issued on 1 October 1991. The new contract to install a secant pile cutoff wall was awarded on 14 April 1992 to Rodio-Nicholson Joint Venture (RNJV). When the new contract was let in April 1992, the Dike 1 work platform was constructed in the configuration shown on Plate A-7. Wall excavation began on 14 October 1992 with the contractors Superdrill (Plate A-8), drilling 34-inch diameter holes on 24-inch centers. Upon completion of drilling, each pile was filled with concrete using the gravity tremie method, creating a continuous concrete wall (Plates A-8 & A-9). A minimum 18-inch contact length between adjacent piles was specified. If this was not accomplished to the satisfaction of the Contracting Officer, conforming remedial piles were drilled as needed. In some cases the 18" overlap requirement was relaxed at the discretion of the Contracting Officer when the piles in question were found to lie within areas of low permeability. The wall was completed on 26 August 1994 and the site restoration was completed 6 December 1994. The total cost of completing the project was $33,535,000.

The maximum flood control pool of 1128 elevation restriction was lifted on 23 January 1995 contingent upon no new evidence of the previous seepage problem.

1.6 <u>Contractor's Organization.</u>

1.6.1 <u>Supervision Organization Rodio-Nicholson (RNJV).</u>

a. Principal in Charge - Mr. G. Dugnani

b. RNJV Executive Committee

(1) Mr. L. Ginetti
(2) Mr. G. Dugnani
(3) Mr. J. Nicholson

c. RNJV Jobsite Management

(1) Project Manager

(a) Ron Triplett 5/92-2/93
(b) Steni Stefani 2/93-8/94
(c) Alexandro Alfonso 8/94-12/94

(2) Field Superintendent

(a) Francesco Cecchini 5/92-6/93

(b) Francesco Sidoti 6/93-12/94

(3) Quality Control/Safety Manager

(a) Bill Call 5/92-3/93
(b) Wes Schmutzler 3/93-12/94

(4) Geologist - Donald Bruce

1.6.2 <u>Subcontractors.</u>

a. Ingersol-Rand - Tom Lovell

(1) Keystone Services - Richard Soppe

(Provided Hammers and Drill Bits)

b. Beaver Lake Concrete - Gene Daniels

(Supplied Concrete)

c. E. Berkley Traughber (EBT)

(Quality Assurance NX Coring and 6-Inch Joint Coring)

d. Engineering Services, Inc. (ESI)

(Survey Control)

e. Greer Excavating

(Landscape Restoration)

1.7 <u>Corps of Engineers Organization.</u>

1.7.1 <u>On Site Personnel.</u>

a. Project Engineer.................. Larry Pitchford
b. Construction Representative...... Tom Tadpole
c. Construction Representative...... John Earwood
d. Geologist........................ Patrick Jordan

1.7.2 <u>Administrative Personnel.</u>

a. Administrative Contracting Officer...... Daniel Clemans
b. Project Manager............ John Balgavy; Sherry Spencer

SECTION 2

2. GEOLOGY

2.1 Regional Geology and Physiography. The Beaver Dam and reservoir areas are located in the Ozark Plateau Province, within the Interior Highlands Physiographic Division. The Ozark uplift consists of flat-lying rocks consisting predominantly of limestone, dolomitic limestone, sandstone and shale with a natural dip of less than 2 degrees southward. The local upland section around the dam is named the Springfield Plateau, the surface of which is developed at about elevation 1500 on a cherty limestone known as the Boone Formation. In the dam and reservoir region the White River has cut the rock to a depth of about 600 feet below the plateau surface. The region is deeply and intricately dissected by streams, resulting in narrow, steep valleys and ridges. Four geologic formations outcrop in the region and all are present at the dam site (Plate B-3). In ascending order, they are the Cotter and Powell in the Jefferson City group, which are of Ordovician age, the Chattanooga formation (Chattanooga Shale and Sylamore Sandstone Units) of Devonian age, and the Boone Formation of Mississippian age. In localized areas faulting has occurred causing the younger (Chattanooga and Boone) formations to be down thrown into the older Cotter/Powell formations.

2.2 Site Geology. In the vicinity of the dam site, the Boone Formation caps the higher ridges and forms the sides of the valleys down to about elevation 1,200. Respectively, beneath this lies the Chattanooga Formation (shale member) and its Sylamore sandstone member. Beneath these and forming the valley walls below elevation 1,180, and underlying the greater part of the valley bottoms are limestones and dolomitic limestones of the Jefferson City Group.

The Boone Formation generally lies (in situ) above the reservoir; however, due to downfaulting of the graben beneath Dike 1, the Boone is the foundation of Dike 1. Being constantly submerged beneath a high column of reservoir water resulted in an acceleration of the usual weathering process (solutioning, chemical weathering, etc) that removes calcium carbonate from limestone creating karstic conditions (voids). A combination of this process, along with colloidal size soil removal from the weathered rock zones, and the natural faulted conditions under Dike 1, resulted in the continued seepage increases that warranted the construction of the remedial cut off wall.

2.2.1 Structural. Beaver Dam lies near the northeast end of a shallow, elongated, northeast-southwest trending structural basin known as the Price Mountain syncline. This basin is often faulted in areas where the downfolding is most pronounced.

The most predominate geologic feature of Dike 1 is created by two NE/SW trending faults traversing through the site near the north end of Dike 1 and the north end of the main embankment (see Plates B-4 & A-12). The interior portion between these faults has been downthrown resulting in a graben that includes all of Dike 1 and 200 feet of the main embankment, extending from approximately dam station 63+00 to 75+00. The graben is bounded by steeply dipping normal faults on either side trending in a northeast-southwest direction. The north-bounding fault(s) tend to dip southeast and the south bounding fault(s) dip northwest. The interior of the graben consists of younger Mississippian aged Boone limestone bounded on each side by Ordovician aged (Jefferson City) and Devonian aged (Chattanooga Formation) strata. Both faults are post-Pennsylvanian and Pre-Cretaceous in age and there is no evidence of recent movement.

The trough (graben) was formed as a result of over 200 feet of vertical displacement. This movement and subsequent breakage resulted in large areas of disturbed material and en echelon faulting particularly in the south fault zone. While some fault planes were healed and loose debris solidified naturally into various forms of breccia with calcium carbonate and other minerals, others remained open and were capable of transporting water, resulting in seepage shortly after completion of the dam. Two attempts were made to solidify this graben area by grouting, once during construction and again in 1968 with marginal success (See Plate B-4 for bottom of construction grout curtain & A-12 for bottom of grout curtain after the second grouting attempt).

2.2.2 Bedrock Stratigraphy. Descriptions of the formations encountered at Dike 1 during secant pile wall excavation are given in ascending order and likewise are shown on the geologic column in Plate B-3.

2.2.2.1 Jefferson City Group. Consisted of unweathered gray dolomitic limestone with thin irregular green to gray shale partings, and occasional greenish gray shale bands up to 2' thick. This group was never differentiated into specific formations during the project and was referred to as Ordovician or Ordovician Dolomite. Because this rock was generally unweathered and competent, it was used as the tie-in rock for the wall at both the north and south ends and occasionally at depths.

2.2.2.2 Chattanooga Formation.

2.2.2.2.1 Sylamore Sandstone Member. Consisted of massive beds of fine-to-medium grained, white sandstone, hard, but with a friable texture. The Sylamore outcropped at several places just outside the graben area with a normal thickness of 3-5 feet. In the south faulted zone; however, there were repeating beds up to 10 feet thick (Plate A-12). It was difficult for the Rodio Super Drill to penetrate the Sylamore Sandstone. It appeared that the air hammer broke the sandstone down into sand and the sand cushioned the impact of the hammer. In addition, the drill holes could not be cleaned properly.

2.2.2.2.2 Chattanooga Shale member. The Chattanooga shale lies conformably upon the Sylamore sandstone. It varies in thickness from 12 to 20 feet and is described as a black-to-brown, carbonaceous, fissile, slickensided, moderately hard shale, with occasional clusters of pyrite.

2.2.2.3 Boone Formation.

2.2.2.3.1 St. Joe Member. The lower member of the Boone Formation is the St. Joe limestone, and described as a non-cherty, gray-to-green, hard, unweathered limestone containing occasional thin seams of shale. The St. Joe was the easiest rock to excavate during the contract.

2.2.2.3.2 Upper (No Name). The upper member of the Boone can be further divided into two types (zones). Both types are composed of calcium carbonate and silica; however, in the uppermost zone the silica appears as an admixture (precipitate) with the calcium carbonate, or as a siliceous limestone, containing pockets and /or lenses of pure limestone (carbonate). Weathering of the siliceous limestone resulted in removal of the limestone (carbonate), leaving a porous, vuggy, soft silica-rich remnant which typically appears as "chalk" mixed with red clay from the totally decomposed limestone (to red clay). However in the lowermost zone of the upper Boone member, the silica is present in the form of thin lenses and/or nodules of pure, hard chert within a massive, gray crystalline limestone. The chert bodies have no vertical or horizontal pattern but are randomly scattered throughout the limestone. Typically, as groundwater flows through joints and fractures in this zone, the limestone (carbonate) is removed in solution, resulting in extensive open and/or clay filled cavities. The numerous caverns (caves) in the area are located in the Boone Formation.

2.2.3 Overburden. With the exception of the granular fill work platform material and the pervious shell of the main embankment all of the soils are residual. This residual material was formed by the decomposition of limestone or dolomite rock into clays varying from red to yellow colored with angular chert fragments ranging from gravel to cobble sized. The depth of overburden averages 20 feet, however, the overburden and intensely weathered rock overlying the Boone Formation may range to depths exceeding 60 feet (see Plate B-8).

2.3 Engineering Characteristics of Bedrock Materials. A total of three hundred seventy eight samples from the Boone Limestone, Sylamore Sandstone and Ordovician Dolomite were tested for unconfined compressive strengths. Strength values ranged from a low of 1,632 to a high of 27,378 psi. The Boone Formation exhibited the greatest strength. Approximately 14% of all cores tested were 15,000 psi or higher. In addition Los Angeles Abrasion Tests were performed on three selected groups of quarry-run samples of the Boone Formation. Results are given in Table 1.

TABLE 1

RESULTS OF TEST OF CHUNK STONE

FIELD DESCRIPTION	LIMESTONE CHERT	SILICEOUS LIMESTONE & CHERT	LIMESTONE
SPECIFIC GRAVITY	2.64	2.37	2.69
ABSORPTION %	.8	4.5	0.6
LA ABRASION %	28.6	28.8	29.2

2.4 Unusual or Unanticipated Geologic Conditions Encountered.

Even though it was known that Dike 1 was highly permeable from numerous seepage exits and piezometer levels that followed the lake fluctuations, it was surprising that the secant pile wall had to be so nearly completed before any noticeable impact was observed. Numerous alternative seepage paths must have existed under Dike 1. The wall appears to have effectively cut off the water seepage from the lake through these subterranean caves and passage ways, many of which may have held and transported water before Beaver Lake was ever constructed. Placing an impervious wall through this area, has been a positive and effective step.

The local ground water regime will adjust to the cutoff wall and some unexpected events may still occur. The ground water situation should be monitored closely at different pool elevations for several years to come.

There were no other geological conditions in the contract which were not anticipated. The locations of the two faults and graben, as well as the lithology of the area, were known and delineated in geologic cross sections (Plate A-12) and boring logs. Large quantities of water entering the borings, as well as unstable drilling conditions were to be expected from reviewing the contract specifications and the exploration logs.

SECTION 3

3. Foundation Explorations

3.1 Investigations Prior to Construction.

3.1.1 Exploratory Borings. Thirty-two exploratory borings were drilled along the upstream crest of Dike 1 and its abutments during the period from April 1986 to October 1988. Boring locations were chosen to either verify structural features detected by geophysical methods or determine the vertical and horizonal extent of features uncovered by previous borings. A plan view of borings is presented on Plate A-11, and a profile of the first 25 borings is shown on Plate A-12.

Extensive investigations were conducted on each of the borings, including soil and rock sampling, diamond core drilling, detailed descriptive logging of rock core, dye testing at zones of drill fluid loss, pressure testing of rock, downhole geophysical logging, inspection with downhole video equipment, and the laboratory testing of rock core samples.

The stratigraphy uncovered by the exploratory borings showed that the northern fault zone extended from station 63+00 at elevation 1,110 to station 66+00 at elevation 880 feet (a vertical offset of 210 feet). The southern fault zone extended from station 70+75 at elevation 894 to approximate station 75+35 at elevation 1,040 (an approximate vertical offset of 146 feet), with considerable disturbance continuing to the southernmost borings, as evidenced by the presence of repeating beds of Sylamore Sandstone and Ordovician Dolomitic Limestones (Plate A-12). Both the northern and southern fault zones contain extensive areas of weathering, cavitation and jointing that would permit seepage under Dike 1. While no significant offsets of the stratigraphy were noted in borings interior to the fault zones (the down thrown portion of the graben), cavities were common and both jointing and fracturing were extensive. Drilling water connections occurred between several borings in this area, which indicated seepage was pervasive along the entire length of the dike and not localized to the fault zones.

3.1.1.1 Dye Testing. Dye testing of exploration boreholes was routinely conducted at zones of complete drill fluid loss. Typically, Rhodamine WT or Fluorescein dye was introduced to the top of the drill hole and clear water was pumped into the hole from 1 to 4 hours. Major seepage exit points were then periodically inspected visually over the next 24 hours. Only boring BE-24, located in the north fault zone, produced a visible dye return in seepage Exits 17 & 18 which were located 900 feet

from the boring (Plate A-6 and A-11). Activated charcoal packets placed at seepage locations however picked up traces of dye indicating that connections existed between the exploratory holes and seepage exits. The results were of limited value in determining seepage paths because dye remained in the seepage piping system months after a tests was conducted. This made it impossible to tell which positive test result was related to a specific dye injection. Control charcoal packets that were placed before dye test often proved positive.

3.1.1.2 Pressure Tests. Pressure test results are shown on Plate B-1. The largest water takes occurred in the fault zones, with no evident reduction with depth. Pressure tests in the Boone Formation, as with other methods of investigation, showed little lateral (between borings), or vertical correlation. This indicated the lack of homogeneity and porous nature of the Dike 1 foundation.

3.1.1.3 Miscellaneous Field Test. Downhole geophysical logging was completed on the majority of the borings. However, the Gamma and Electric logs displayed little similarity between borings other than the overburden/rock contact interface and the appearance of the Chattanooga Shale Formation. Caliper logs were more useful in confirming cavities shown on the boring logs. Temperature deviations from the normally expected stratification of the water column indicated water movement through fractures from the lake.

3.1.1.4 Rock Strength Test. Three hundred and seventy eight samples were tested at the Southwestern Division Laboratory for density (specific gravity) and unconfined compressive strength. The compressive strengths varied from a low of 1,632 psi to a high of 27,378 psi, with an overall average strength of 10,084 psi. The average specific gravity of the samples was 2.64. Rock strength test results are shown on Plate B-2.

3.1.1.5 Downhole Camera Observations. Southwestern Division Laboratory personnel performed both "down-looking" and "side-wall scanning" observation and video taping of the borings with a downhole, black and white camera. Numerous open cavities, channels, joints, and intensely fractured zones were encountered, mainly in the upper, cherty Boone Formation. Open cavities having depths beyond the focus of the camera were a common occurrence. Subsurface flows through channels in rock were apparent in several borings where suspended fines could be seen moving rapidly. Downhole camera observations were most valuable in confirming boring log data.

3.1.2 Geophysical Investigations.

3.1.2.1 The Objectives of Geophysical Investigations.

a. Detect, map and monitor seepage beneath Dike 1.
b. Delineate the geologic structure beneath Dike 1.
c. Determine a logical pattern for subsequent exploratory drilling and additional piezometer locations.

3.1.2.2 Geophysical Methods Used. The geophysical methods employed at the Dike 1 investigation included the following:

Methods	Primary Applicability	Secondary Applicability
Self Potential (SP)	Seepage Detection, Mapping & Monitoring	Geologic Mapping
Electrical Resistivity (ER)	Geologic Mapping	Seepage Mapping
Seismic	Geologic Mapping	
Borehole Fluid Conductivity (BC)	Seepage Mapping	
Magnetic Profiles (M)	Geologic Mapping	
High-Resolution Seismic Reflection	Geologic Mapping	
Ground-Penetrating Radar	Geologic Mapping	Detection of high water content zones
Microgravity	Geologic Mapping	Embankment Integrity

3.1.2.3 Chronology of Geophysical Investigations.

Date	Event
Nov 1984	Waterways Experiment Station (WES) personnel visit Beaver Dam for a site inspection prior to planning geophysical program.
Dec 1984	WES proposal for a geophysical seepage assessment program at Beaver Dam transmitted to SWL for consideration.

Jan 1985	Southwestern Division, SWL, and WES personnel meet at Beaver Dam for review of proposed geophysical program and site inspection.
Feb 1985	Funds to support geophysical program transmitted to WES. SWL personnel establish survey lines and install SP electrodes.
Mar 1985	Four WES personnel perform major geophysical field work as outlined above.
Apr 1985	WES personnel perform preliminary interpretation of survey data and recommended new piezometer locations (30 new piezometers were installed as recommended).
Aug 1985	WES personnel forward recommendations for exploratory boring locations (32 exploratory borings were drilled based primarily on the geophysical data).
Sep 1985	USGS and WES personnel conduct ground probing radar survey along Dike 1.
Jan 1986	Program review meeting at Beaver Dam to assess the status of all on-going work related to seepage assessment. Date set for finalization of WES input in project. Borehole conductivity measurements acquired.
Feb 1986	USGS personnel perform additional ground probing radar study at Dike 1.
Mar 1985-1987	SWL personnel acquire SP data and forward to WES
Aug 1986	High-resolution seismic reflection survey conducted. Sp "Low Pool" survey conducted. Electrical Resistivity soundings conducted.
Oct 1986	WES/SWL co-authored paper on Dike 1 seepage investigations presented at the annual meeting of the Assc. of Engineering Geologist (Llopis, Butler, Deaver and Hartung 1986)
Feb 1987	SP "high pool level" survey conducted.

3.1.2.4 Summary of Investigations and Conclusions. Geophysical investigations conducted at the site were useful in determining the locations and strikes of the bounding faults of the graben

beneath Dike 1 (Plate B-5). Seismic refractions surveys were used to map the extent, both laterally and vertically, of the weathered rock (Boone Formation) underlying the downstream berm of Dike 1 (Plate B-6). Results of SP surveys indicated that seepage was pervasive and generally did not occur in well defined conduits, or zones, except at the south fault, nor was limited to the two fault zones. Geophysical tests, particularly SP, indicated that some concentrations of seepage occurred in the upper elevations of the south fault zone and across its entire length. Temperature and conductivity tests inferred seepage was coming from the lake and suggested possible flow paths. Significantly, the Seismic Reflection & Ground Penetrating Radar Survey marked numerous possible faults or fractures and cavities in the interior of the graben which may have served as preferential conduits for seepage, particularly if the fractures were solution-widened as some appeared to be. Fractures or faults in the interior of the graben were not previously suspected but were later confirmed by exploratory drilling. The Self Potential (SP) test indicated that the northern fault zone seemed tight, as there was no geophysical test evidence that inferred seepage across or along it. The geophysical results, drilling results, and dye tests were used to construct an integrated methods seepage map, Plate B-7.

Results from geophysical interpretations, and the map were used to plan piezometer and exploratory borings which further refined the knowledge of the seepage regime. As an example, exploratory Hole BE-24 (Plates A-12 & A-6) made a direct connection with seepage Area 1 (Exits 17 & 18) during dye testing.

In conclusion, the drilling investigations and geophysical testing were invaluable in developing the cutoff wall concept and design specifics. The sum total of information was found to be accurate in depicting the subsurface seepage regime. Wall construction also produced information that supported or confirmed the accuracy of data obtained through the drilling investigation and geophysical testing. As an example, during construction of the cutoff wall, Piles 1 through 40 (Sta 62+00 to 62+80) on the north side of the north fault were excavated dry but piles south of 62+80 generally produced abundant water. Seepage Area 4 also dried up after the wall was constructed past the north fault and adjacent areas confirming both geophysical and drilling exploration data.

3.1.3 <u>Historical Piezometer Data.</u> The location of Dike 1 piezometers is shown on Plate C-1. Historically, all piezometer levels on Dike 1 have responded directly to the lake level.

Piezometer charts in Plates C-3 through C-13; representing water levels for individual piezometers from January 1991 to January 1995, show Dike 1 water level/lake level correlation. The fact that a direct connection existed between the lake and practically every piezometer on Dike 1 inferred the pervasiveness of the seepage paths, thus supporting the need for a continuous impermeable barrier, such as a secant pile wall, to block all of the seepage through Dike 1.

Plate C-14 shows a contour map of piezometer levels in 1991 representing a low lake elevation of 1114. The most notable feature was the 1105 elevation contour that generally paralleled the south fault zone and extended east beyond Dike 1 for several hundred feet, eventually terminating at the artesian piezometer near Seepage Area No. 2 (See Plate A-6). This entire area was hydraulically 9 feet or less below lake level, and the artesian piezometer only 2 feet below lake level, thus showing a clear, direct path (only a two foot head loss) from the lake to the artesian piezometer and Seepage Area 2 located 500 feet away.

3.1.4 Historical Seepage Data. The location of seepage exits are shown on Plate A-6. The quantity of seepage (Plates C-17 through C-20) generally remained stable from January 1991 to March 1994, excluding rains, when it began to decline due to the effects of the secant pile wall that was under construction. The seven historical events associated with Dike 1 seepage are given in Table 2.

TABLE 2

DATE	EVENT	RESULTS
1966	Initial Reservoir Filling	Seepage detected in Area 1 (800 GPM)
1968-1971	Remedial Grouting	Seepage reduced to (500 GPM)
1975	Reconnaissance Report	Seepage Detected in Areas 2, 4 & 6
Dec. 1984	Record Pool (1130.4)	Muddy Flow in Seepage Area SA-1, Exit 9; Seepage Areas SA-3 and SA-4 Increased
June 1993	Wall Completed to Pile 278	Seepage Area 4 Dries Up

Mar. 1994	Pile Wall Complete Except from Pile 503 to 636	Higher Elev. Seepage Dried up During Grout Stabilization of Area
Aug. 1994	Pile Wall Complete	Seepage Above El. 1040 Dried up

3.2 Investigations During Construction.

3.2.1 Concrete Coring Data. Table A-19 provides the location and summary of concrete quality control testing performed on each of the piles and pile joints which were cored during the contract by the subcontractor, E. Berkley Traughber & Associates. In addition 3 other Piles (16, 136 & 138) and Joint 16/17 were cored by the Little Rock District Drill Crew.

Coring was done with a 1500 Failing drill rig. Grouting of completed core holes was done with a Chem-Grout colloidal mixer. Joints were cored using a 6 by 7 inch core barrel and piles were cored using a NX core barrel until the hole became deep enough to insert a NQ wire line barrel (normally of 5 ft. length). Joints were typically drilled to 30 feet depth and piles were drilled to full depth plus 5 feet into the bedrock. After drilling was completed, the core hole was tremie grouted with a 1:1 grout mix, plus a non-shrink admixture (Interplast-N) until filled. If grout shrinkage occurred, the hole was topped off at a later date.

Due to the nature of the contract which stated that no quality coring would be necessary until a continuous 100 foot section of wall was completed, Rodio-Nicholson initially planned not to complete any 100 foot section until late in the contract or possibly at the end of the job. They were confident that the concrete placed would meet the criteria for the contract (primarily a 3000 psi strength) and would be of a quality not to negate the design intent. As a result of Government concern over concrete quality, Little Rock District personnel performed the first concrete quality coring on the secant pile wall, from 15 November 1992 to 20 November 1992. Piles 16, 138, 136 and Joint 16/17 were cored in the same manner as prescribed in the contract except a 4X5 inch core barrel was used to core the joint, and the holes were left open (grouted up later by the contractor). It was found that all of the sampled piles contained moderate to severe honeycombing, water bleeding conduits, some vertical fractures that seem to not be made by sample drilling, and segregation. In addition, Pile 82 (cored by the contractor) had a rod drop from 149.2'(bottom of solid concrete in the pile) to 152.4'(top of natural rock below pile). This caused concern both

to the government and the contractor, about the quality of concrete placed in the piles and resulted in the contractor performing quality coring on a regular basis throughout the remainder of the contract. All cores taken were logged by Corps representatives and E. Berkley Traugher personnel.

The initial concrete cores were tested at the Corps of Engineers Southwestern Division Testing Laboratory and verified that the concrete met and typically exceeded the contract strength requirements of 3000 psi. The laboratory; however, stated that only relatively intact samples were tested. Samples that were broken, badly segregated or otherwise of poor quality, were discarded because they were not testable. It was decided that the best way to test the permeability of the concrete wall would be to pressure test the piles that had been cored. Visual attempts to decide if concrete core was acceptable or not, always lead to the Government and the Contractor being of opposite opinions. Consequently, the water pressure test became the primary method of determining concrete quality.

Pressure testing was conducted with the packer inserted near the top of the hole (40 psi for piles, and 15 psi for joints) and held 5 minutes. If no significant water take occurred, then no further tests were performed. If the pressure test resulted in a significant water take then additional tests were made, moving the single packer progressively deeper, until the pervious part of the pile was isolated.

Since neither the Government nor the Contractor were completely satisfied with the concrete quality, both investigated the problem. There were varying opinions, but the first clue came after a careful examination of the concrete core. It was found that many of the abrasions and imperfections in the concrete cores were linear and running vertically. Later after further examination, it was found that not only were the imperfections vertical (parallel to the length of the core) but some were also arced horizontally (across the core). This arcing corresponded to the diameter of the tremie pipe (10 inches) being used to place the concrete. In some cases, a partial outline of the tremie pipe could be seen in the imperfections of the concrete core. Since the concrete tremie pipe apparently had created part of the problem, concrete placement came under closer scrutiny. It was found that the tremie sometimes became blocked, especially near the top of the piles. This occurred because the weight of the concrete in the tremie pipe and placement hopper was not sufficient to overcome the weight of the concrete already in the pile. As a result the tremie pipe had to be raised and lowered repeatedly to keep the concrete flowing properly. It was

also discovered that the bad areas of the core contained no aggregate, very little bonding agent (cement) and consisted primarily of fly ash. The up/down movement of the tremie pipe had caused a pumping action resulting in the lighter fly ash being segregated, leaving a poorly cemented fly ash zone in the track of the tremie pipe. This was confirmed by drilling Pile 136 a second time (by the contractor this time) at an offset 1 foot to the left of the pile center. The results were a much higher quality concrete on the edge of the pile than that previously cored from the center. Contractor personnel then designed a new concrete mix without fly ash, but with more cement and other admixtures which resulted in the quality of the concrete being greatly improved. A more complete explanation of this problem is given in the Construction portion of this report.

This first core drilling effort also indicated that the contact between the bottom of the piles and the top of natural rock usually (in over 80% of cases) had a debris zone that resulted in rod drops of one to four feet. The reason for this was never actually determined. It could have been caused by improper cleaning of the bottom of the hole or fall-in from the sides of the pile. Nevertheless, it persisted throughout the construction with an occasional exception such as the excellent concrete/rock contact observed at Pile 4. The rubble zone at the bottom of the concrete wall is believed insignificant in the long term performance of the concrete wall because all of the piles were seated several feet into unweathered solid rock. Taking into consideration that water will probably never reach the bottom of the wall, and if it does, enough piles have made a tight seal with the rock beneath to prevent any seepage water from traveling very far.

Plate A-19, provided by Rodio-Nicholson, lists all of the piles that were contractor cored for quality control during the job, along with pressure testing results and grouting data. None of the piles or joints took enough water to cause any concern. Core holes at two Pile Joints (73/74 and 400/401) had the higher water takes; however, a water penetration rate of 2-3 gal of water per minute in a 7-inch hole, 24 feet deep, under 15 psi is practically impermeable. No specific treatment was made on either of these two pile joints. Generally the joints were so well bonded that it was difficult to find the joint contact. The piles that were drilled, were also tight and any water takes in the cored piles were attributed to the rubble zone below the wall or the drill bit exiting the side of the pile into a joint or natural rock. During coring, Pile 250 made a direct connection with Pile 258 that had previously been cored and left open through the rubble zone below the wall. Even though it appeared

obvious where the connection had taken place between the piles (as observed during coring) several water pressure tests were made moving the single packer progressively deeper into the hole until the rubble zone was definitely verified as the area of water take. It should be noted that the contractor's test (Plate A-19) on Pile 250 indicates the amount of water take after the rubble zone was sealed off by grouting. This was done in order to pressure test the concrete portion of the pile.

3.2.2 Dye Tests. Three dye tests were conducted by the government during construction of the Rodio-Nicholson wall at Beaver Dam (18 June 1993, 28 September 1993 & 10 February 1994). The injection points were exploratory holes BE-9 & BE-2 & piezometer ME-8 respectively (Plates C-1 & A-11). Dye tests were performed near incomplete sections of the wall to allow free access of lake water to the dye injection point. If the test proved positive it would be repeated again after the wall was completed. Fluorescein dye was used in each of the three tests. After placing dye at each of the injection points, activated charcoal packets were placed at various seepage exits and regularly retrieved and replaced to detect trace amounts of dye. None was ever conclusively detected and no further dye test were attempted.

3.2.3 Seepage Data. As construction proceeded completing the wall, the location of seepage exits remained constant, no new exits emerged as original seepage exits were blocked off. However, by the time the wall was completed all seepage areas above approximately elevation 1040 had dried up. Seepage locations were frequently checked for muddy flows, along with quantity (gpm) of seepage from the French Drain, Flumes 1 & 2, and the South Ravine Weir (Plates C-17, C-19 & C-20). The Artesian well (Gauge) was checked for water level changes during most of the contract (Plate C-18), along with simply listening for a change in the noise level of water rushing into the water well (Plate C-1). Seepage from all sources eventually collected in a natural pond approximately 300 feet east of the powerhouse road. This was regularly checked for any discoloration up until the time when the contractor began discharging drill water into the north ravine. This data is summarized in Plate C-16, along with explanations for anomalies (rain caused high flows, etc.) in the remarks section.

The first seepage to be influenced by wall construction was in seepage Area 4, located near the north end of Dike 1 (Plate A-6). On 8 June 1993, this seepage area was observed to be dry (Plate C-16). This date corresponds to a time interval (16 May 1993 to 28 June 1993) in which 13 piezometers began a rapid decline at

the north end of Dike 1 (Plate C-2). Most of the northern third of the wall had been completed to Pile 278 or dam station 67+82. Plates A-14 (1 June 1993) and A-15 (15 June 1993) show progress completed on the cutoff wall at their respective dates. None of the piles drilled immediately before 8 June 93 indicated any unusual conditions during the drilling (excessive water) or the concreting (large placement takes). It appears likely that Seepage Area 4 dried up because the secant pile wall had gradually blocked all of the alternative seepage paths from the lake as the wall became continuous past the area. The decline in the water table was not specific to just seepage exits or Seepage Area 4, but wide spread as indicated by piezometer drops throughout the northern third of the dike (Plates C-3 to C-5).

The start of the last major seepage reduction began on 29 March 1994, which corresponded to dramatic drops in a number of piezometer levels (Plate C-2) from 15 March 1994 to 22 April 1994. At this time, the wall was completed except for the interval between Piles 503 to 636. Plates A-16, A-17 & A-18 show the progress made on the wall for 14 March 1994, 4 April 1994 & 11 May 1994, respectively. The material that had to be excavated (with the Superdrill) in constructing this portion of the wall was unstable at higher elevations and required consolidation (downstage grouting) before it was stable enough to drill. Consequently, it was the last section of the wall completed. The piles in this wall section were normally drilled and grouted first to approximately 65 feet (completed early April 1994), and then approximately 90 feet (completed June 1994) before they could be completed to their required depth (185 feet) and concreted to form the completed wall. From 14 March 94 until 1 June 94, most of construction consisted of downstage grouting. This period of grout stabilization resulted in the greatest seepage reduction (Plates C-17 through C-20) during any one period of wall construction. The grout (mainly 1:1 mix) not only stabilized the top 90 feet of weathered material, but also filled cavities and voids, thus effectively blocking most seepage even before the actual wall was finished.

Initially, about 29 March 94, the most noticeable seepage declines occurred at higher elevations; the artesian piezometer (G31) - fell below ground level for the first time; the French drain - dropped from 15 gpm to 3 gpm; the south ravine weir - dropped from 180 gpm to 130 gpm; and the seepage in Area 3 was reduced substantially (Plate C-16). At this point most of the primary piles in the uncompleted wall portion had been stabilized to approximately 65 feet deep (el.1065). A horizonal seepage line had developed, and above this line (elevation) all seepage

had dried up. As grouting continued this line (elevation) continued to drop. By the time the grouting was completed (June 1994) Seepage Areas 2, 3, 4 & 5 were dry and Seepage Area 1 was dry except at lower elevations. The artesian piezometer was 30' below ground level, the French drain was dry, the sand boils in Seepage Area 1 were reduced and the south ravine weir flow was down to approximately 15 gpm (Plate C-16). Gradually, seepage continued to be diminished and by 17 Sep 1994, all seepage areas above elevation 1040 were dry and Flume 1 (el. 1030) was flowing only 4.7 gpm (Plates C-16 through C-20).

The seepage in the area between Piles 503 and 637 and the area in the south fault zone, was wide spread as evidenced by the progressive lowering of the phreatic line as the grouting blocked off seepage paths at progressively deeper depths.

3.2.4 Piezometer Data. Piezometer locations are shown in Plates A-6 and C-1. Ground water levels were monitored by 56 automated piezometers during the secant pile wall construction at Beaver Dam. Water levels were measured automatically at 4 hour intervals except during occasional outages of the automated system. Charts in Plates C-3 through C-13 were generated from this data. There were only two major changes of piezometric head pressure during the contract, both were drops, with the first occurring from 16 May 1993 to 28 June 1993 and the second from 15 March 1994 until 22 April 1994 (Plate C-2). The drops in piezometer levels occurred simultaneously with the drying up of surface seepage exits (Plates C-16 through C-20). There was also a false rise in piezometer levels that occurred during the time period from 11 March 1993 until 24 March 1993. This was caused by mechanical adjustments.

The first major piezometer drops (16 May 1993 to 28 June 1993) occurred after most of the wall from dam station 62+00 to 67+82 had been completed (Plates A-14 & A-15). All piezometers north of piezometer P-13 except C32.2 were affected (see Plates C-1 & C-3 through Plate C-5), with some water level reductions up to 30 feet. These reductions occurred simultaneously with the drying up of Seepage Area 4. There were no indications immediately prior to this water level reduction that any significant amount of seepage had been blocked off during construction. Likewise, there was no erratic behavior from any of the Dike 1 piezometers. They followed the lake level as usual indicating no change in the ground water regime. This leads to the conclusion that seepage must have been quite pervasive and the foundation rock was so permeable that virtually the entire section of wall had to be completed before any noticeable piezometric level reduction took place.

Piezometers P-17, B27.9B & B29.1B dropped initially during the first piezometric reduction but rose again during October & November of 1993 (Plates C-3 & C-4). These three piezometers are located on the extreme north end of the job and are probably fed from rainfall infiltration from the highlands located northeast of the dam. It was also possible that since only about 1/3 of the wall was completed, an increased lake head (Plates C-3 & C-4 show an increase in lake level at this time) allowed lake water to flow around the south end of the incomplete wall, causing rises in piezometer levels. This option is considered less likely because other piezometers further to the south that should have been affected by southern end-around seepage did not rise.

The second major drop in piezometric levels occurred from 15 March 1994 to 22 April 1994. Piezometric levels dropped across the entire length of Dike 1. Piezometer charts (Plates C-6 through C-13) illustrate the generally rapid piezometric level declines with some drops of over 40'. At this point in construction, the wall was totally completed except for the wall segment between Piles 503 and 636, which was completed last due to instability problems in the area (Plates A-16 through A-18). It was necessary to down stage grout this area to a minimum of 90 feet, in most cases, before normal drilling and concreting techniques could be performed to complete the wall. It was during the grouting phase of construction that most piezometric levels were dramatically reduced. The drops occurred during a time period when the lake was increasing, indicating the effectiveness of the cutoff wall.

Plates C-14 & C-15 show piezometric contour maps of Dike 1 ground water levels for October 20, 1991 (before the contract was started) and October 20, 1994 (2 months after completion of the wall), with the lake elevation being 1114 for both dates. Three observations can be made that indicates the effectiveness of the wall:

1. The highest contour on the 1994 map is approximately 1080 while the highest contour on the 1991 map is approximately 1105 elevation. A piezometric head reduction of approximately 25'.

2. The high piezometric contour area connecting the lake to the artesian piezometer on the 1991 map is no longer present in the 1994 map.

3. The contour lines are more regular and spaced further apart on the 1994 map than the 1991 map, meaning the hydraulic gradient has decreased.

While the initial period of piezometric reduction occurred during March and April of 1994, water levels in many piezometers continued to drop several months after the completion of the wall (Plates C-3 through C-13). The contour maps drawn are therefore preliminary in nature and may not reflect the final effect of the secant pile wall.

3.2.5 Ground Water Encountered During Construction. Nearly every pile drilled encountered water (except Piles 1 - 40 on northern end). This water was due to 3 possible sources: a) the work platform granular fill consisted of a highly permeable poorly graded gravel that often extended below lake level; b) the southern portion of the wall cuts through the pervious shell zone of the main embankment and c) the natural weathered Boone Formation transported water from the lake to the drill holes. All of the exploratory holes drilled prior to the contract produced water, except on the north side of the north fault zone. Therefore, encountering large inflows of water while constructing the secant wall piles was expected. Table 3, gives some of the water and stability conditions that resulted from the highly permeable conditions encountered beneath Dike 1 during construction.

TABLE 3

Pile No.	Condition
1-40	Dry, had to pump water to lubricate bit
84	Large cavity
90	Large amounts of water produced
114	Large amounts of water produced
120	Unstable, cavities
122-144	Connected during drilling
164-214	Connected during drilling
258-294	Connected during drilling
292	Took water; used as a sump to dispose of water
318	Water observed flowing through hole
334	Water observed flowing through hole
336	Took water; used as a sump to dispose of water
402	Large amounts of water produced
406	Large amounts of water produced
439	Large amounts of water produced
452	Water observed flowing through hole
499	Cavities
502	Large amounts of water produced
582-586	Connected during drilling

605	Connected w/upstream ditch
612	Sink hole developed on platform
614	Water observed flowing through hole
619-617	Connected during drilling
621-600	Connected during drilling
632	Connected with adjacent piles
634	Sink hole developed on platform
668	Large cavity
690	Sink hole developed on platform
693	Sink hole developed on platform
717-719	Holes filled with grout simultaneously
738	Blew off last 3' of platform while being drilled, cracked soil stabilization concrete

Note: the term "connected" refers to the situation of drilling or working in one boring with air, water or grout and a subsequent flow to another open boring, thus showing that an open conduit connected the two shafts.

As can be seen from Table 3, there are connections, cavities and weathered zones extending from the north fault zone south to the southern end of the wall that did transport water before the wall was completed.

3.3 Post Construction Performance of the RNJV Concrete Cutoff Wall During High Pool Elevations. After the completion of the RNJV wall and the initial draft of the Beaver Construction Completion Report, a sustained high pool elevation (above elev. 1128) occurred between the dates 8 May 1995 and 26 June 1995. An intensive investigation of observable seepage and piezometric levels was performed, detailing the effect of the completed wall. The investigations indicated no adverse or unanticipated conditions occurred either in surface seepage patterns or the ground water regime at high pool elevations. Piezometer levels and seepage volumes generally remained stable or showed marginal increases due to rainfall runoff/infiltration. The seepage test further indicated that the cutoff wall is effective in controlling seepage for lake levels to EL. 1130 (top of flood control pool), resulting in an unconditional return to the pre-1986 lake management and inspection conditions. The memorandum, CESWL-ED-GS, Subject: Trip Report Concerning the Performance of the Concrete Cutoff Wall During High Pool, is presented in Appendix E.

A piezometric contour was made of the Dike 1 area during the high lake pool elevation (EL. 1128.8) that occurred on 7 June 1995 (Plate C-21). If compared to the low pool elevation (EL.

1114) in Plate C-15 (20 October 1994), which was also after completion of the RNJV wall, it can be seen that the contours on both are virtually the same. This indicates that the elevation of the lake pool level had little effect on ground water pressures after the wall was completed. Comparing these two piezometric contour maps to Plate C-14 (lake level of el. 1114) before the RNJV wall was constructed, an obvious change in both the elevation and gradient of contours can be seen. The Pre-wall contours (Plate C-14) being both higher in elevation and closer together. This leads to the conclusion that high lake pool elevation did affect the Dike 1 foundation negatively before the wall was constructed, resulting in increased seepage, muddy flows, piezometric rises etc., but is no longer the case after the wall was completed.

Since September 1994, at which time the total flow (seepage and rainfall runoff) at Flume 1 was 4.7 gpm (see paragraph 3.2.3), a very significant change occurred in the total flow at Flume 1. On 21 August 1995, after a number of continuous weeks of no rainfall which historically masked the true seepage flow below Dike 1, it was reported by the former Beaver Project Engineer that no visible seepage was emerging at any point from Dike 1 and thus no flow was occurring at Flume 1. This marked the first time in the 29 year history of Dike 1 (April 1966) that seepage had not been experienced at Dike 1. The pool level on 21 August 1995 was el. 1119.8.

SECTION 4

4. SPECIAL DESIGN CONSIDERATIONS

4.1 Contract Requirements.

a. The Contract required the construction of a concrete cutoff wall of approximately 206,000 square feet with a length of 1475 feet and a depth ranging from 80 to 185 feet through embankment, overburden consisting of weathered rock, and a hard, permeable zone of limestone foundation rock with compressive strength up to 27,000 psi.

b. The concrete cutoff wall was to be located between dam stationing 62+00 and 76+75, approximately 65 feet upstream of the centerline of Dike 1.

c. The cutoff wall was to be constructed of interconnected panels, columns (secant piles), or a combination of the two.

d. The contractor was responsible for maintaining stability of the excavation for the cutoff wall.

e. The concrete cutoff wall was to have a minimum width of 24 inches and/or have a minimum joint contact between adjacent panels or secant piles of 18 inches.

f. The concrete for the cutoff wall was to have a minimum compressive strength of 3000 psi at 28 days.

4.2 Contractor Design/Construction (Soletanche-Rodio-Nicholson Joint Venture).

a. The first contractor selected for the design/construction of the cutoff wall was Soletanche/Rodio-Nicholson Joint Venture (SRNJV) under Contract No. DACW03-89-C-0041.

b. SRNJV proposed to construct the cutoff wall using interconnected panels. SRNJV chose a rock mill or hydrofraise for excavation of the wall. After constructing the platform, SRNJV constructed reinforced concrete guidewalls to be used in control of the hydrofraise excavation. The location of those guidewalls were as follows:

Station 62+00 to 65+00	Length 300 feet
Station 71+00 to 74+45	Length 345 feet

The guidewalls were 36 inches deep and 12-16 inches in width located on both sides of the centerline of the cutoff wall approximately 41 inches apart, with a top elevation of 1130.

c. SRNJV attempted to excavate through the overburden and underlying rock with the hydrofraise, but was unsuccessful and the contract was terminated.

4.3 <u>Contractor Design/Construction (Rodio-Nicholson Joint Venture).</u>

a. The second contractor selected to perform the construction of the cutoff wall was Rodio-Nicholson Joint Venture (RNJV) under Contract No. DACW03-92-C-0039.

b. RNJV used the existing platform and guidewalls previously constructed by SRNJV as well as their own. RNJV chose to construct the cutoff wall using the secant pile method (Super Drill machinery) with 34 inch diameter secant piles constituting the cutoff wall spaced 24 inches on centers with generally a width of 24-34 inches (See Plates A-9 & A-10) but never less than 10 inch overlay between piles.

c. The location of the project site was in a remote area and, therefore, consideration of the following affected design and construction:

(1) RNJV had to mobilize and stock their own supply facility as the nearest large city where drilling supplies were available was Tulsa, OK, located 2.5 hours away.

(2) RNJV brought on site their EC. 80 drill rig capable of using a 34-inch drill bit with 32-inch drill rods, and constructed on site a second drill rig capable of drilling a 34 inch hole and holding 180 feet of drill rods within the mast.

(3) The contractor had to erect a concrete batch plant on site.

(4) The contractor had to provide 13,700 volt phase to phase electrical power lines to be constructed from three miles away to operate a bank of nine air compressors.

(5) The contractor erected a building for Ingersol Rand/Keystone Drill Services to rebuild drill bits and hammers since the drill bits and hammers were unique for this project.

d. Stability of the Excavation

(1) The contractor had to take into consideration the design and the construction for maintaining stability of the cutoff wall excavation.

(2) RNJV mobilized bentonite holding tanks and constructed slurry pits to use bentonite as a means of providing stability of the excavation performed in the overburden. There was a risk, however, of losing the bentonite slurry into cavities in the highly weathered rock as well as in the upper granular material of the platform.

(3) Once construction was underway, it became necessary to modify the contract and use Portland Cement grout to stabilize the highly weathered rock zone. (This is discussed further in Section 5 of this report.)

SECTION 5

5. CONSTRUCTION

5.1 Equipment and Plant Layout.

5.1.1 Drilling Equipment. Drilling of the secant piles to form the cutoff wall was performed by using down-the hole hammers mounted on two drill rigs. The first drill rig (Rig No. 1) was mobilized from Italy and the second (Rig No. 2) was designed and built on site by RNJV. Rig No. 1 was equipped with a rod loader capable of inserting 30 feet of drill rod at a time into the drill rod stem. Rig No. 2 was constructed with a lattice-type mast that could support two vertical rod changers each containing 60 feet of drill rod stem as needed. Each drill rig had 100 foot masts mounted on a crawler crane. Rig No. 1 was mounted on a Link Belt 318 and Rig 2 was mounted on a Manitowoc 4100 crane. Following is a list of attached equipment:

- Power swivel rotary heads (Soilmec EC-80, Watson)

- Hydraulic Power Pack installed at the rear of the crawler cranes to provide power for the rotary head, hydraulic stabilizers, and hydraulic pistons controlling inclination of the mast.

- Thirty-two inch diameter drill rods with 11-7/8" inner air passage. Each drill rod was 30 feet in length.

5.1.2 Down-the-Hole Hammers and Drill Bits.

a. The majority of the piles were drilled by two models of down-the-hole hammers. The main down-the-hole hammer used was the Ingersol-Rand DHD130. Also used was the Sandvik XL24. Both hammers operated on the same principle as they were pneumatically driven and mechanically rotated by a rotary head mounted atop the mast of the drill rigs. During drilling, the piston was the only moving part of the hammer. The movement of the piston was controlled by the flow of compressed air and the force from the piston was transmitted directly to the drill bit, which in turn, crushed the rock and soil at the hole bottom. Compressed air moved the piston in a two-stroke movement and exited through the drill bit, transporting drill cuttings to the surface of the hole in the annular space between the drill stem and the shaft walls.

b. Drill bits consisted of a 34-inch diameter relatively flat surface with embedded carbide buttons. The drill bits rotated slowly (1-10 rpm). A cluster drill bit (Ingersol Rand CD24-5)

was occasionally used and worked on the same pneumatic drive principle as described above. The hammers moved up and down at a rate of 900 cycles a minute. The cluster bit consisted of five conventional downhole hammers, with a diameter of 8-inches, shrouded in a casing having a diameter of 24 inches. The downhole hammers were arranged in a pattern that, when the cluster drill was rotated, yielded a flat surface, having a diameter of 34-inches. This bit was very effective in cutting the Sylamore Sandstone, increasing the drilling rate from 2-feet/hour (IR DHD130) to 6-feet/hour. While its production rate was fast there was a problem keeping verticality using this bit, as a result it was normally used only for the specific task of drilling Sylamore Sandstone.

5.1.3 Auxiliary Equipment.

a. Crawler Crane (Linkbelt LS-338) used in concrete placement and general use.

b. Hydraulic Crane (rubber-tired Loraine LR-230E) used in general operations and maintenance.

c. Trackhoe (Linkbelt LS-2800CH) used for digging and maintaining drainage ditches and sediment ponds.

d. Front End Loader (Trojan 1700Z) - maintenance.

e. Backhoe (Case 580K) - maintenance.

f. Two tractor/trailer rigs used to haul drill cuttings from drill site to disposal area.

g. Subcontractor furnished concrete mixer trucks.

h. Air Compressors - Nine air compressors were electrically operated and were capable of delivering 960 CFM each and supplying a constant air pressure supply of 300 psi. The nine air compressors were divided into two banks of four compressors each with one spare compressor that supplied air to the other two banks if needed. The average air pressure supplied to the drill rigs was 150-170 psi.

i. Water Pumps - Two-inch to six-inch pumps were used to pump water to the drill rigs while drilling was underway on "non-water" producing holes and also to pump water away from the drilling operation into a drainage system.

j. Diverters - Drill cutting diverters were fabricated and used in drilling operations to dispose of drill cuttings into tractor/trailer rigs and/or temporary disposal storage areas.

5.1.4 Plant Layout.

a. An air compressor building housed the nine air compressors and electrical panel boxes.

b. Ingersol Rand/Keystone Drill Services constructed a building and office to perform drill bit and hammer maintenance operations.

c. A welding operations and maintenance building was constructed and used for fabrication of diverters and other support equipment as well as maintenance.

d. A general maintenance shop was used for repairing pumps and other drilling equipment.

e. Since the project was located in a remote area, the contractor had to maintain a general supply office.

f. The contractor maintained an office trailer for office staff of seven personnel and a field office trailer for field personnel, including the field superintendent and QA/safety manager.

g. A concrete batch plant was located just off site and operated by a subcontractor, Beaver Lake Concrete.

h. The contractor maintained a disposal area for drill cuttings in a quarry located approximately 3/4 mile south of the cutoff wall site on Government Property.

5.2 Platform Preparation and Surveys.

5.2.1 Platform Preparation.

a. The platform was constructed by SRNJV under Contract No. DACW03-89-C-0041. The platform was constructed to el. 1130 of random fill consisting of clay/gravel. The width was approximately 75 feet and length approximately 1550 feet.

b. Under Contract DACW03-92-C-0039, RNJV placed approximately one foot of crushed rock on the upstream platform surface and sloped the rock surface back to the guidewall and the adjacent downstream collector ditch, at elevation 1130, to alleviate muddy

conditions caused by surface water. Sloping the platform surface away from Beaver Lake to a drainage ditch system also prevented bentonite slurry and other contaminants from entering the lake.

5.2.2 Surveys.

a. RNJV employed a subcontractor, Engineering Services, Inc. (ESI), to lay out the location of the cutoff wall on the platform and locate the center of piles to be drilled. ESI established control points which allowed RNJV's Quality Control organization to maintain the cutoff location and location of piles to be drilled. Occasionally, ESI would have to reestablish control points which had been destroyed.

b. The contractor performed weekly elevation surveys on Dike 1 to determine if the dike showed signs of settlement. No settlement was discovered throughout the construction of the cutoff wall.

c. When drilling first commenced on a typical hole, the contractor used transits located at 90 degree angles to plumb the drill rig masts, and used lasers located at 90 degree angles to maintain verticality of the drill masts while drilling operations were underway.

5.3 Drainage Provisions.

5.3.1 The contractor sloped the platform 1-2% from Beaver Lake to a drainage ditch running parallel to Dike 1 located at the intersection of the Dike 1 slope and the platform surface. The drainage ditch ran the full length of the cutoff wall and had small settling ponds at several locations, with a 20' x 40' settling pond at the North end of the platform. Once the suspended solids had settled out, the water then flowed into Beaver Lake.

5.3.2 The above system, however, did not work once heavy Spring rains flooded the platform and rapid runoff did not allow solids time to settle; therefore, drainage became discolored and flow into Beaver Lake had to be stopped. Also, the excavation process brought large amounts of ground water to the surface as lake water passed through the granular platform material and foundation rock and into the holes being drilled. Ultimately the contractor constructed a new drainage system by building an overhead bridge across Highway 187 which supported two six-inch pipes, and then pumped the water across the road into the northern ravine below Dike 1. The ravine was lined with rip rap baffles and silt screens to prevent erosion and allow sediment

removal. The drainage water then flowed into a large settling pond where solids were allowed to settle out and the drainage water then flowed down the ravine approximately 1/2 mile before emptying into the White River. The Arkansas State Pollution and Control Board monitored the system by examining test specimens of the drainage water.

5.4 <u>Excavation.</u>

5.4.1 <u>Overburden Excavation and Stabilization.</u>

5.4.1.1 The RNJV notice-to-proceed was issued on 19 May 1992 with mobilization continuing through 30 September 1992.

a. It should be noted that the contract with RNJV was a design/construct contract. RNJV had submitted a Technical Proposal stating the methods to be used in excavation and construction of the cutoff wall which the Corps of Engineers accepted.

b. The contractor, RNJV, began overburden excavation and stabilization in the area where SRNJV had previously constructed guidewalls, Station 62+00 to 65+00. In this area, RNJV excavated the overburden between the guidewalls, downward to the top of the weathered rock zone. The layout and profile of the cutoff wall is shown on Plate A-4. The logs of preconstruction exploratory borings are shown on Fig. A-12. Once the top of weathered rock was reached, RNJV then backfilled the excavated area between the guidewalls with 1000 psi concrete to stabilize the excavation. The contractor placed 682 cu. yds. of 1000 psi concrete which was of insufficient compressive strengths to use the Super Drill operation. The 1000 psi concrete was removed and replaced with 3000 psi concrete which allowed use of the Super Drills. RNJV performed overburden excavation in the manner described above from Sta. 62+00 to 65+00 and one small section of guidewall at Sta. 65+60 to 65+80 which had been constructed by SRNJV. The area Sta. 71+00 to 74+45 where guidewalls also had previously been constructed by SRNJV, was unusable by RNJV except for a few feet either side of Sta. 72+00. The remaining guidewalls that were unusable in this reach were removed by RNJV. RNJV decided for the remaining overburden excavation of the cutoff wall, to backfill the excavated trench which was approximately 6 feet wide and excavated to the top of weathered rock, with 3000 psi concrete monolithically in lengths varying from 12 feet to 41 feet. The overburden excavation and stabilization was completed on 10 March 93. The following equipment was used in the overburden excavation construction:

- Linkbelt LS218 with a clam grab
- Hyundai 290 Trackhoe
- Case 580K Backhoe
- 7-Ton Chisel

Bentonite slurry was used in excavation of the overburden trench between Sta. 74+16 to 74+28 and approximate Sta. 75+52 to 76+77. The overburden trench excavation and backfill, 3000 psi concrete, provided a permanent concrete casing for the unstable soil above the top of the weathered rock zone. It was the contractor's intent for the overburden trench excavation to penetrate the weathered rock zone to form a seal into it and, for the most part, this was successful. Therefore, for the majority of the overburden excavation and stabilization, the contractor excavated a trench through the overburden to the top of the weathered rock zone. This trench was approximately 6-feet wide with varying depths from 12' to 72' deep. This provided a "concrete casing", for drilling through the overburden. The contractor performed the overburden excavation in stages; once the overburden stabilization concrete reached 2000 psi in an area, rock drilling excavation (with the Super Drill) could proceed while other areas of the overburden trench were being completed.

5.4.2 Rock Excavation and Stabilization.

5.4.2.1 Rock excavation began on 14 October 92 when RNJV drilled secant pile No. 16 to a depth of 80 feet using the EC-80 Super Drill with a 34-inch drill bit. The excavation began by drilling through the concrete filled overburden trench, through a weathered rock zone and then penetrating a firm rock zone to the bottom of the hole. There was a total of 739 secant piles drilled to form the cutoff wall. The majority of secant piles drilled (485) were drilled at rates of 15-25 feet per hour and reached required depths with minor difficulties. The average penetration through various rock types and 3000 psi concrete is given in Volume 2, Appendix G, Part 3. Occasionally, holes had to be cleaned out after drill rods were removed. This was accomplished by air lifting or using clams. The contractor first drilled primary holes that were spaced four feet apart and then drilled secondary piles which overlapped the primary piles a minimum of 10 inches on either side. See Plate A-9 and Plate A-10 for drilling schematic. See Appendix G, Volume 2 for drilling data. For convenience, RNJV divided the Secant Pile Wall into four segments. These segments were primarily based on the stabilization treatment required to complete the drilling of the holes in each area with the Superdrill. The segments, the corresponding piles & stabilization techniques are listed below:

TABLE 4

AREA	PILES	STABILIZATION METHOD
A	1-496	Generally None Required*
B	497-638	Grout Downstaging
C	639-686	Generally None Required
D	687-738	Grout Downstaging

* Some concrete downstaging, especially in North Fault Zone

Two hundred fifty-four (254) secant piles encountered unstable conditions in the weathered rock zone which had to be overcome by a "downstaging" process to stabilize the holes to where the holes could be drilled to required depths, remain open, and be backfilled with concrete.

a. Downstaging Stabilization of the Excavation - The Super Drills used an air pressure of 150-170 psi while drilling the 34-inch diameter secant piles. Also used was water to control dust when water was not encountered in the hole while drilling. The drilling process forced air, water and drill cuttings to the surface where the cuttings were placed in tractor/trailers and hauled to a disposal site which was an old rock quarry. Once drilling began in the weathered rock zone below the concrete filled overburden trench excavation, the contractor had to closely monitor the amount of rock cuttings blown to the ground surface and the depth of drilling. If the amount of drill cuttings was substantially more than the theoretical amount indicated as the depth increased, the contractor knew the drill hole was becoming unstable. This was also evidenced by air loss, although air loss was also an indication of cavities within the weathered rock zone. When the contractor could not continue to drill because of air loss or more rock material being removed than the theoretical volume, drilling was stopped and the drill rods, hammer and bit were withdrawn from the hole. Generally, the hole then collapsed to a variable extent. The hole was sounded using a metal chain tape and then filled with concrete to the top of the hole. This procedure is called downstaging. Once the concrete had reached 2000 psi or higher compressive strength, the hole was redrilled and the above process was repeated. If the hole was highly unstable in the weathered rock zone, the contractor used the process of downstaging 2-5 times in the same hole being drilled. The process of downstaging, using concrete, formed a concrete casing through the unstable rock zone. Once drilling reached the firm, stable rock zone which lay beneath the weathered rock zone, drilling excavation would be continued to

the bottom of the hole. As stated previously, the contractor encountered 254 of 739 secant piles which had to be downstaged to provide a stabilized hole.

This procedure of downstaging was very costly to the contractor and the magnitude had not been accurately anticipated during the bidding process. Also, once the contractor began drilling the 185-foot secant piles in areas of the South fault zone, approximate Station 72 to 75, the downstaging process using concrete, which was presented and approved in the Contractor's Technical Proposal, caused major problems. Drill rods, hammers, and bits became "hung" in the holes when some of the concrete downstaging material shifted because of unstable rock conditions. Also, unstable rocks collapsed against the drill rods in the hole causing the drill rods to become fastened and unable to turn. The contractor had used the downstaging process using concrete on approximately 80 previous unstable piles when it became apparent that this process would not work on the 185-foot piles. The contractor skipped the area and moved to Sta. 75+75 at the southern end of the cutoff wall. As drilling began on these 130 foot piles, the underlying material located beneath the concrete filled overburden trench, comprised of granular fill and highly weathered rock, could not be excavated using the Super Drills in a safe manner. This was because the granular material extended to the platform surface outside the overburden trench and drilling caused areas of platform surface to collapse as the underlying material was being removed by the drilling process.

The contractor proposed using a grout downstaging method to stabilize the highly unstable weathered rock zone. The contract was modified to use grout instead of concrete and the result was successful. This method is discussed in paragraph below. The contractor used grout downstaging to stabilize the weathered rock zone on 174 holes. The downstaging summary results are shown in Appendix G (Vol.2). These results are listed as Redrill Footage and Non-Spec Concrete Quantities and Grout Quantities.

b. Grout Downstage Stabilization of the Rock Excavation. When it was realized that concrete downstage stabilization was not effective in the highly unstable weathered rock in Area "B", grout downstaging of the weathered rock zone was attempted and worked very successfully. The following is a discussion of the grout downstaging method adopted:

(a) Grout mixes, with water to cement ratios of 1:1; 0.75:1; and 0.5:1, were used, with the majority placed being 1:1.

(b) Grout was introduced into the drill hole through a tri-cone roller bit fastened to the drill rods using the Super Drills.

(c) The Super Drill excavated the secant pile hole with a hammer and 34-inch drill bit with carbide buttons drilling through the concrete filled overburden trench and into the highly unstable weathered rock zone. Once drill cuttings exceeded the theoretical volume of hole per foot of drilling, the drilling was stopped, rods pulled and the drill hammer and bit were replaced by a tri-cone roller bit through which grout was discharged. The drill rods and tri-cone bit were lowered into the drill hole to the depth of hole collapse and the Superdrill forced the tri-cone bit into the collapsed material, and grout was discharged through the tri-cone bit until the grout reached the surface of the hole. If the grout take exceeded expectations, a lower water to cement ratio was used until the grout remained at the hole surface. The drill rods and tri-cone bit were removed and the hole was again refilled to the surface with grout. Approximately 99% of the grout used was a 1:1 mix. Many of the secant pile drill holes had to be grouted a second time at a lower depth to provide a stable weathered rock zone. Occasionally, a hole would have to be grouted 3 or 4 times but this was rare. The downstaging grout method provided a stabilized weathered rock excavation, thus allowing the Super Drills to complete the holes into the firm rock zone. The holes remained stable until filled with concrete forming a portion of the concrete cutoff wall.

c. Pressure Grouting Consolidation Test Section. During the construction of the cutoff wall between Sta. 66+00 and Sta. 69+00, a highly fragmented weathered rock zone was encountered approximately 50 feet down from the platform surface. The Super Drill lost air in this stratum and drilling was extremely difficult. The contractor proposed to the Government to pressure grout the stratum by drilling two lines of 7-inch diameter holes, four feet apart, located each side of and parallel with the cutoff wall. The 7-inch diameter holes were located one foot outside the upstream and downstream concrete filled overburden trench for a distance of 100 feet (Sta. 66+00 to Sta 67+00). This was to be a trial or test section in an attempt to pressure grout the highly weathered rock zone. The following equipment was mobilized for this effort:

- Hydraulic crawler rig with a 7-7/8" diameter tri-cone roller bit and set up to drive a 7" diameter casing.

- Grout mixer and pump

The area was pressure grouted but not very effectively since the highly weathered rock zone was not as permeable as first thought. Fourteen secant piles were drilled in the test area after the pressure grouting, but still had difficulties. Downstaging with concrete proved more effective in this area.

5.5 Quality Control of Drilling Excavation.

5.5.1 Secant Pile Locations. RNJV used a subcontractor, Engineering Services, Inc. (ESI), to survey the location of the cutoff wall and establish control points. Angle iron rails were installed on the overburden concrete filled trench at el. 1130 to use as guide rails. The angle iron rails were surveyed in place and fastened securely to the concrete surface. At four foot intervals, metal tags were welded to the angle iron rails showing hole numbers. Holes were drilled in the angle iron rails every two feet, thus giving the center of every secant pile to be drilled (see Plates A-9 & A-10 for drawing schematics). This procedure precisely located the top of the secant piles to be drilled. A diverter was fastened to the angle iron rails by bolts whose locations had been surveyed to establish pile centers. The Super Drills were moved over the diverters and drill bit placed in the diverter, thus securing the drill bit and drill rods at the ground surface on the center of the secant pile to be drilled.

5.5.2 Drilling Plumbness Control. Once the drill bit was placed and secured on the hole center, the drill hammer rods were plumbed by using transits located at 90 degrees to each other. After 5-10 feet of drilling into the concrete filled overburden trench, the drill rods were plumbed with lasers located at 90 degrees to each other and each laser had striking receivers wired to monitors which were located at the top and bottom of the drill masts. Monitors were installed in the Super Drill cabs where operators could maintain constant verticality of the drill masts and rods with hydraulic inclinometer.

5.5.3 Verification of Hole Depth. The stiffness of the 32-inch drill rods maintained a relatively straight hole. Once the holes were completed and the drill rods, hammer and bit were removed, the holes were checked with a metal tape chain to verify depth to hole bottom. If the measured depth indicated drill cuttings remained in the hole, the cuttings were removed by air lifting or by a clam. Air lifting was a cleaning process by which air was forced downward through the inside of a 6-8 inch diameter pipe and allowed to escape at the pipe tip just above the drill cuttings. As the air moved upward, it created a vacuum within the pipe and the suction removed the drill cuttings to the ground

surface where the cuttings were placed in the quarry disposal area. To use air lifting, the drill holes needed to be over 60 feet in depth to be effective. Occasionally, a clam had to be used if larger size rocks had fallen into the hole from the weathered rock zone and could not be lifted to the surface by the air lifting procedure. Once the hole was cleaned, it was rechecked with the steel chain tape. If cuttings were still present, the above procedure was done again until the hole was clean and ready to be checked for verticality.

5.5.4 Verticality Determination. The contractor, with assistance from the Government, designed and constructed a "verticality device" which, in essence, was a submersible plumb-bob. The device was a steel barrel, 30 inches in diameter, and four feet in height with rigid cable mounts protruding outward from the outside of the barrel against the 34-inch diameter hole walls. This configuration insured that the verticality device remained centered within the hole at all times while being lowered down the hole. A floating target attached by cable, independent of the steel barrel with the cable passing through an eyelet located on top center of the verticality device, remained centered within the pile with reference to the verticality device. The pile hole had to be filled with water to within 10-15 feet from the surface. As the verticality device was lowered to the bottom of the hole, the floating target centered itself with respect to the hole bottom once the cable was in tension. The target was positioned approximately 10'-15' down from the top of the hole on the water surface. An X-Y axis was established at the ground surface of the hole and the position of the target could then be compared to the center of the hole at the ground surface using the X-Y axis and lowering a plumb-bob to the center of the floating target. The center of the hole bottom with respect to the center of the hole at the surface therefore could be determined. Also, the hole was checked at various intermediate depths using the same procedure.

To verify that the reverse plumb-bob was accurate, dry holes were checked by lowering a plumb-bob from the surface to a target on the hole bottom which was centered, and positioning the plumb-bob with high intensity lights and binoculars. The results obtained using this reverse plumb-bob in a dry hole were recorded, water was placed in the hole until it reached a ten to fifteen foot depth from the surface, and then the verticality of the hole was checked using the submersible plumb-bob apparatus. Results showed only a one-quarter to one-half inch deviation which was considered satisfactory. Contractor Quality Control personnel would often check each other by performing verticality checks independent of each other on the same hole. Also,

Government inspectors would perform the test randomly to insure verticality results were satisfactory. Also, by checking the verticality results at various depths, ratios could be developed to comparatively check the results as the 32-inch drill rods in a 34-inch diameter hole kept the hole straight.

The verticality results are shown for each hole in Appendix G (Volume 2) which also gives overlapping arc lengths. The lowering of the verticality barrel to the hole bottom assured a 34-inch hole, and that it was clean. If the verticality results did not meet contract specifications, an extra overlapping (conforming) pile had to be drilled and filled with concrete. The contractor had to drill 24 of the extra piles to correct piles which did not meet contract requirements. The data from these piles are shown in Appendix G (Volume 2).

5.6 Concrete Placement and Quality Control Testing.

5.6.1 Concrete placed in the cutoff wall had to have a minimum unconfined compressive strength of 3000 psi. The contractor took 3 concrete cylinders on every pile. Compressive strengths varied from 3200 psi to 5700 psi with the average being approximately 4200 psi. The batch plant was operated by a subcontractor, Beaver Lake Concrete. The subcontractor maintained two concrete truck mixers at all times and was able to add various admixtures, as well as hot and cold water. He also had a laboratory on site. The batch plant was capable of producing 200 cu. yds. per hour.

5.6.2 The approved concrete mix design at the beginning of the project was as follows for one cubic yard:

Sand	1480 lbs plus or minus, depending on moisture
Gravel	1660 lbs
Cement	368 lbs (Type 1)
Flyash	132 lbs (Type C)
Water	225 lbs plus or minus, depending on moisture content of sand
ProKrete N	25 oz (Super Plasticizer)
Rheobuild 561	9 oz (Water Reducer)

The concrete specifications were as follows:

Maximum w/c ratio of 0.5
Concrete slump range of 7-9 inches
A water reducing admixture shall be used
A 28-day compressive strength of 3000 psi
Flyash (30% by volume) was permitted

Concrete tests performed by the contractor showed the above mix to have an initial set time of 23 hours to 26-1/2 hours.

5.6.3 RNJV used the tremie pipe method of placing concrete in the secant piles. Concrete was placed in the excavated holes immediately after cleaning the holes and checking the verticality. The tremie pipe was 10 inches in diameter and was in 10-foot sections. Concrete was placed in a 1.5 cu. yd. funnel-shaped hopper at the top of the hole with the tremie pipe attached below the hopper to the bottom of the hole. The tremie pipe connections were watertight and the tremie pipe and hopper were held in suspension by a steel plate with hinged retention flaps above the hole. A service crane was fastened continually to the hopper and tremie pipes. When concrete placement began, the tremie pipe was raised approximately one foot off the bottom of the hole. A non-collapsible plug was placed into the tremie pipe prior to concrete placement. The plug forced water out of the tremie pipe as the concrete was placed and proceeded downward. During concrete placement, the tremie pipe was kept embedded in the fresh concrete for a length of 10 to 30 feet. The concrete placement proceeded until the hole was filled with concrete.

5.6.4 The contractor requested to change the mix design of the concrete to delete the flyash. The following mix design was approved:

Sand	1108 lbs plus or minus, depending on moisture
Gravel	1660 lbs
Cement	585 lbs
Water	292 lbs
ProKrete	12 oz
Rheobuild 561	9 oz
Air	3 oz

The above mix had a faster set time and; therefore, drilling adjacent holes next to those previously filled with concrete, could be done at an earlier date.

5.6.5 Quality Control of Concrete Mixing and Placement.

- Mill certificates were required for all cement used.

- Aggregates were tested by performing gradation tests and tests performed at the SWD laboratory to meet ASTM requirements.

- The batch plant had a fully automated batching system and uniformity tests were performed on the truck mixers periodically.

- Concrete cylinders were taken on each pile. Unconfined compression strengths were obtained for 3-day, 7-day, and 28-day breaks. All 28-day unconfined compressive strength exceeded the minimum 3000 psi required.

- Slump tests, air content tests, and concrete temperatures were taken for the first two trucks on each pile. Occasionally, the above tests were performed on subsequent trucks if needed.

- NX/NQ quality assurance coring of random selected piles was performed. Also, random selected joints of the pile overlap were cored using a 6-inch barrel. Eighty percent of the core holes were water pressure tested. All holes were backfilled with grout. The test data is shown in Plate A-19.

5.7 <u>Problems.</u>

5.7.1 <u>Aggregates.</u>

a. Sand - Flooding of the Arkansas River caused the contractor to haul sand from an approved source in Oklahoma on several occasions.

b. Gravel - The contractor first obtained gravel from a source in the Kings River at its intersection with US Hwy 62. Occasionally, the gravel either could not be obtained because of environmental restrictions or it did not meet specifications, thus crushed limestone had to be used from a quarry near Avoca, Arkansas.

5.7.2 <u>Cement.</u> Because of cement shortages, cement, on occasion, had to be obtained from a supplier in Kansas.

5.7.3 <u>Platform Sinkholes.</u> Sinkholes (platform collapses) developed in the platform during drilling operations at several locations adjacent to the cutoff wall. One sinkhole developed on the lakeside of the cutoff wall, Sta. 66+00. Other sinkholes developed at several areas on both sides of the cutoff wall, Sta. 75+00 to 76+75. The sinkholes were caused by drilling operations. The platform was constructed of granular material and during drilling operations when excess material was removed, the granular platform material flowed downward and into the holes being drilled, resulting in the sinkholes developing to the ground surface. Once the contractor started closely monitoring

drilling activities and stopped drilling when more than the theoretical amount of cuttings were being removed, the sinkholes stopped.

5.7.4 Drainage. The contractor's drainage system was inadequate as presented in his technical proposal and had to be modified. (See paragraph 5.3. for discussion.)

5.7.5 Downstaging. The contractor had to perform downstaging of holes during excavation of the weathered rock zone in excess of that anticipated in his Technical Proposal. A discussion of downstaging is presented in paragraph 5.4.2.1. of this report. Problems arose when concrete downstaging did not work and the contract had to be modified to use grout downstaging (reference paragraphs 5.4.2.a.& b. of this report). The concrete downstaging method was not satisfactory because concrete could not penetrate the weathered rock zone sufficiently to form a stabilized zone around the pile when redrilled. When grout was used, sufficient penetration into the weathered rock zone stabilized the area surrounding the hole to be drilled and drilling excavation could be performed without collapse.

5.7.6 Concrete Placement. The contractor chose the tremie method of concrete placement to construct the concrete cutoff wall. The concrete mix design approved for use contained approximately 30% flyash. The contractor elected to use constant tremie pipe movement, jerking the tremie pipe upwards to facilitate concrete placement, rather than placing concrete in a hopper at sufficient height above the ground surface to cause the concrete to flow out of the tremie pipe against a 10-foot to 30-foot embedment in the concrete. This action of constant tremie pipe motion allowed water to escape from the densifying concrete, bleed along the pipe, thus carrying some of the flyash to the surface. Concrete cores indicated water blister voids and segregation because of the tremie pipe movement. Concrete cores within the same pile but outside the areas of tremie pipe movement showed satisfactory concrete. To overcome this difficulty, the contractor chose to change mix designs (see para. 5.6.4) and provide an acceptable mix design deleting the flyash and increasing the cement content, and lowering the sand content. This resulted in no segregation or voids in the area where tremie pipe movement was being performed. This action was approved as the results were satisfactory.

5.8 Site Restoration.

5.8.1 Demobilization began on 1 September 94 and continued into October. Equipment and materials were removed and stored in Rogers, Arkansas until moved to another location or sold.

5.8.2 The subcontractor, Greer Excavation, moved onto the site in October 94 and completed landscape restoration on 6 December 94. The Corps of Engineers Park was restored by rebuilding the asphalt road, constructing concrete curbs, placing riprap, hauling topsoil and planting trees. Also, an extra 651 feet of guardrail was installed along Hwy 187 and the park by modification because of safety concerns.

5.8.3 Greer Excavation also dressed the quarry which had been used as a disposal area and cleaned the road surface across the Beaver Dam.

5.8.4 Greer Excavation at the request of the Corps of Engineers placed a clay blanket with a minimum of one foot in thickness, extending from the lake side of the concrete cutoff wall to the Highway 187 embankment (main embankment and dike one) from station 70 to station 77. The clay blanket consists of a red "CH" material and varies in thickness from one to fourteen feet as the contractor's drainage ditch was also filled with the "CH" material. The area between the cutoff wall and Dike 1 (station 62 to station 70) consists of a gravelly clay which ponded water on several occasions during construction caused by heavy rainfall. Much of the area was used during construction as settlement ponds. Therefore the entire area landside of the cutoff wall and the main embankment consists of a clayey material which should prevent lake water above elevation 1130 from entering the subsurface.

5.9 Safety Concerns.

5.9.1 Safety was given top priority on the project by both the contractor and the Government.

5.9.2 Supervisor's safety meetings were held each week rather than the contract's monthly requirements. Tool box safety meetings were held each Wednesday with mandatory attendance by all field personnel.

5.9.3 All employees were required to wear eye protection and ear plugs as well as safety steel toed shoes and hard hats at all times.

5.9.4 The contractor provided incentives for safety milestones (such as jackets, T-shirts, etc.) as awards for safety action and suggestions.

5.9.5 The Government stopped drilling activities when sinkholes developed, as drilling excess material caused the granular platform material to flow downward into the holes being drilled , creating a safety hazard. Once the contractor proposed downstage grouting to stabilize the area being drilled and closely monitored the volume of the drill cuttings; the stop work order was lifted and drilling resumed safely.

5.9.6 Only personnel with keys could access the air compressor building. Only Designated personnel could access the drill bit repair and maintenance shop. All welding was performed by certified welders or someone under their close supervision. All welding was approved by a certified welder.

5.9.7 A safety eye wash facility was provided and it was the policy of RNJV that personnel were taken to a physician when injured, even though the injury was minor. One accident involving a knee injury resulted in 371 lost days, otherwise the safety record would have been considered very good.

SECTION 6

6. Possible Future Problems.

6.1 Wall Extension. One possible future problem has already been addressed. The upper part (45-50') of the RNJV wall does not tie into impervious material at its southern terminus (Plate A-13). A grout curtain cutoff running downstream (90° to the wall) from the RNJV wall and tying into the random fill portion of the main embankment was completed between 15 May 1995 and 21 July 1995. The Beaver Dam Cutoff Wall Extension Grout Curtain Completion Report is included in Appendix D.

6.2 Southern End of Wall. A second potential problem also involves the south end of the wall. The Super Drill, like other air hammers uses air to exhaust cuttings and to run its internal hammer which can and did result in air traveling several hundred feet (Appendix F, Photo 25), possibly opening up passages and cavities that were previously clay filled. While the north end of the wall ties into very competent solid rock (top to bottom), and the wall constructed protects the length of Dike 1; the foundation rock under the Beaver Dam Main Embankment at the southern terminus of the wall may have been affected negatively. The Ordovician Dolomitic Limestone on the south end of the job is

usually competent (even though it may be fractured), however it is fifty feet below ground. The contractor also reported, but never actually verified, a possible cavity below the Sylamore Sandstone, which overlies the Ordovician Dolomite. The piezometric levels near the southern terminus of the wall should continue to be monitored.

6.3 <u>Change in Groundwater Regime.</u> A third potential problem may exist in the interpretation of ground and surface water data. Making Dike 1 virtually impermeable has significantly lowered the ground water elevation just downstream of Dike 1, thus lowering piezometric levels, etc. It may take months for a new, clear hydrologic picture to develop. In the interim if something unexpected occurs, such as a seepage area becomes active again or a higher piezometric level develops in an area, other possibilities should be considered instead of assuming that a breach in the wall has occurred. The highland area near the north end of Dike 1 may be capable of transferring water to seepage areas or causing rises in piezometer levels under conditions such as heavy rains.

6.4 <u>Possible Cavities South of Wall.</u> A fourth potential problem was detected during the geophysical survey of Dike 1. Some of the geophysical methods indicated a possible cavity zone under the Beaver Dam Main Embankment approximately 400 feet south of the southern terminus of the cutoff wall. Generally speaking if a problem does develop it is likely to be at or near the southern terminus of the RNJV Wall under the Beaver Dam Main Embankment. Again this area should be monitored closely for several years.

6.5 <u>Future Sinkhole Potential.</u> A fifth potential problem is that the pile drilling action is known to have created numerous sinkholes through discharging excess drill cuttings that result in cavity formations in the pervious shell of the Main Embankment, the work platform, and the highly weathered rock foundation. The subsequent concreting of the shafts and/or concrete filling of the sinkholes is believed to have refilled any possible cavities (natural or drill action caused). However, it is possible that arching action in the granular material, weathered foundation rock, along with inadequate flow of concrete during shaft tremieing, may have left cavities along the wall. These cavities, if present, could collapse at a future date and result in sinkholes or depressions in the work platform surface.

SECTION 7

7. Lessons Learned.

7.1 Requests For Proposals. For any Request for Proposal (RFP) type contract where a Technical Engineering Evaluation Team has carefully reviewed proposals concerning highly technical or state-of-the-art type work, a member of this team should be included on the negotiation team during final bids to insure that nothing is retracted from a contractor's original proposal that would possibly render the proposal inadequate or deficient toward fulfilling the requirements of the contract with the Government.

7.2 Mutual Understanding. Government RFP contracts have recently become quite common, particularly with international contractors in more technical areas, and much effort should be expended to make certain the contractor has a true understanding of the terminology and intent of the specifications. This is necessary because of a language barrier that sometimes creates different meanings or understanding of words or phrases. During the proposal review stage, clarification should be sought on any statement or statements submitted in the proposal that is not clear to the reviewer, especially concerning important technical issues. Mutual understanding or clarification of these pertinent issues could prevent a costly modification or claim during or subsequent to the contract.

7.3 Superdrill Excavation. Since the Superdrill uses compressed air for cutting extraction and hammer operation, it should not be used in unstable material unless an adequate stabilization method is used. Without stabilization, excessive material may be discharged from the excavation creating cavities and voids.

7.4 Grout Downstaging. Future contract specifications involving the use of down-the-hole-hammers should consider "Grout Downstaging" to restore stability when excavating through weathered rock and granular soil material or in general when encountering unstable soil and rock conditions.

7.5 Concrete Downstaging. A minimum concrete strength of 3000 psi should be used for concrete downstaging if an air hammer is to be used for excavation.

7.6 Variations. Exercise caution in allowing the contractor to vary construction procedures other than in areas where the Corps has adequate knowledge or experience. The contractor should not be allowed to circumvent specifications.

7.7 Drainage. Ensure that the contractor submits adequate drainage system(s) in his Technical Proposal, or specifically require it in the contract specifications.

7.8 Tremie Pipe Concrete Placement. Field experience and concrete testing, from this project, indicated that fly ash should not be allowed in the concrete mix design if the contractor proposes using the tremie pipe method unless additives or other procedures are required that prevent fly ash/aggregate segregation.

7.9 QA Coring/Testing. Specifications should be written in a manner to ensure quality control coring and testing of the concrete is performed on a continuous and timely basis, and consideration should be given to specifying pressure testing of cored holes.

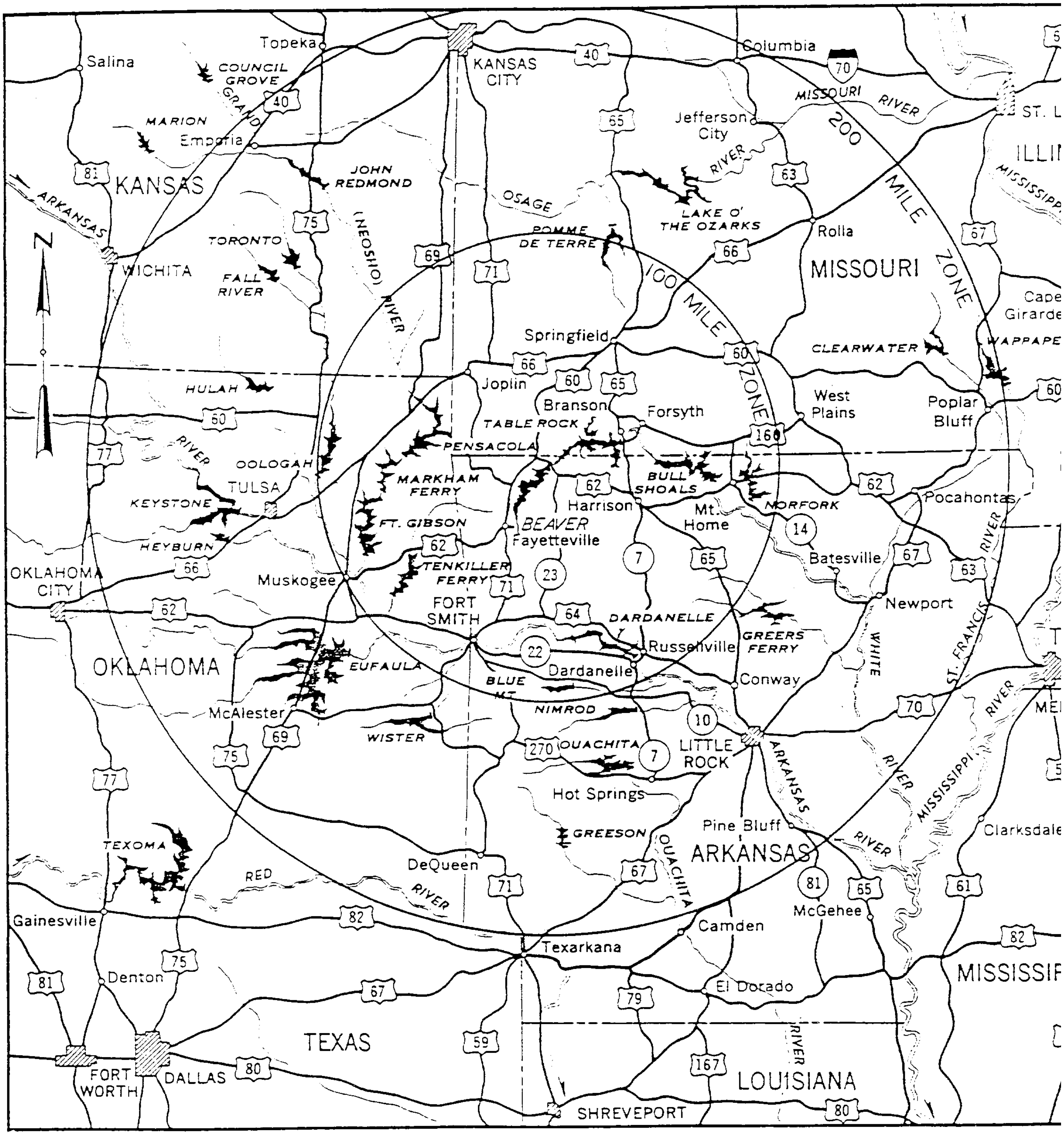
BEAVER DAM COMPLETION REPORT

APPENDIX - A

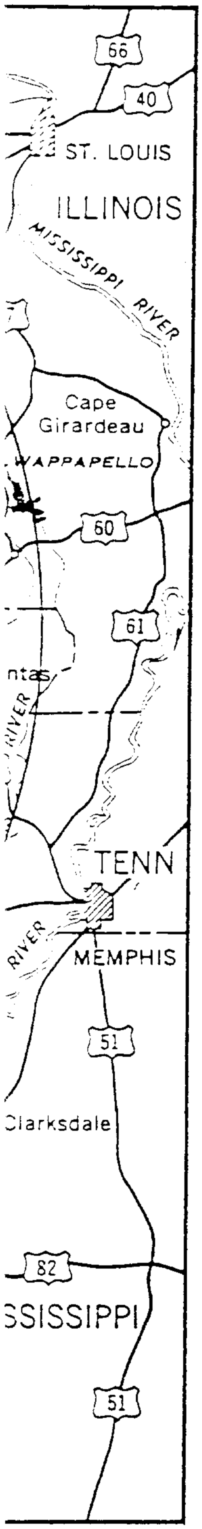
APPENDIX A: CONTRACT & GENERAL DRAWINGS

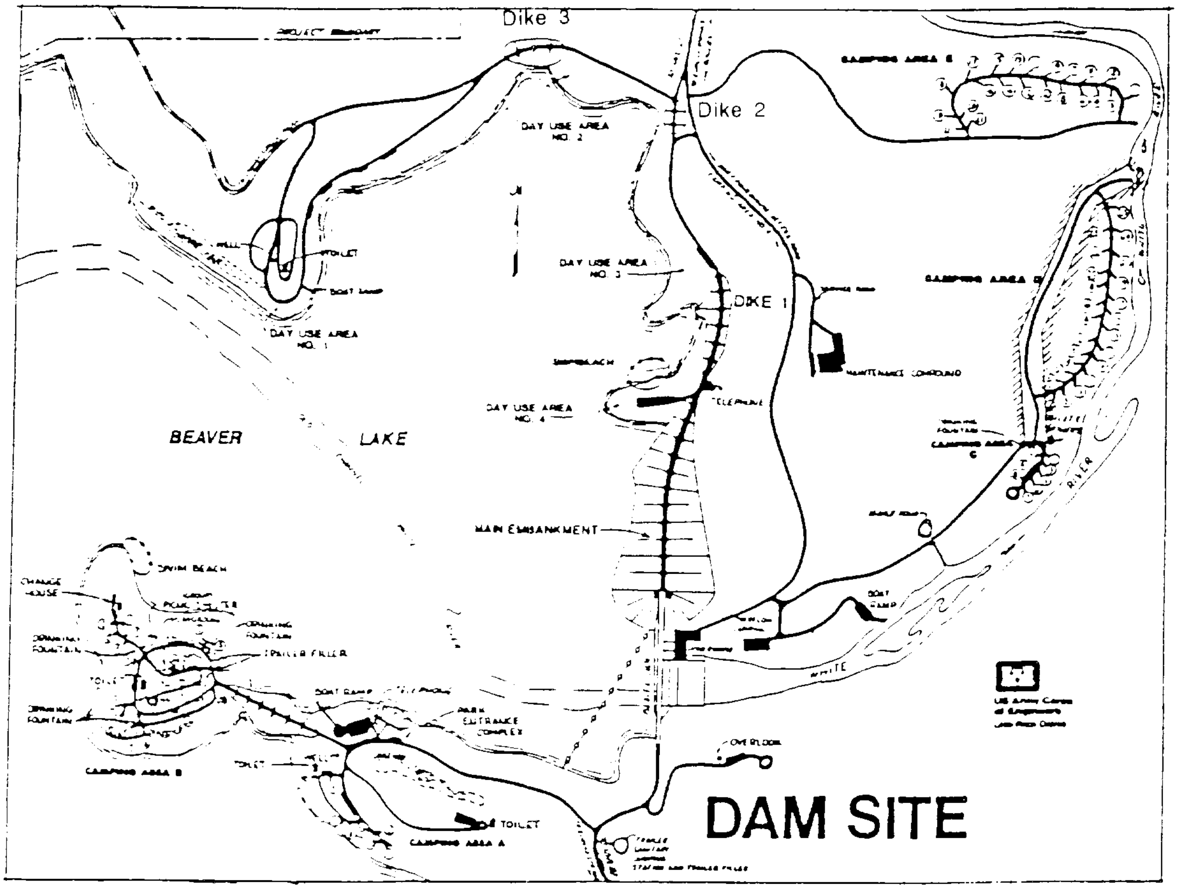
List of Plates

BEAVER DAM



LOCATION MAP


66
40
ST. LOUIS
ILLINOIS
MISSISSIPPI RIVER
Cape
Girardeau
WAPPAPELLO
60
61
TENN
MEMPHIS
RIVER
51
Clarksdale
82
SSISSIPPI
51


Dike 3
Dike 2
DAY USE AREA NO. 2
DAY USE AREA NO. 3
DIKE 1
DAY USE AREA NO. 1
DAY USE AREA NO. 4
BEAVER
LAKE
MAIN EMBANKMENT
SWIM BEACH
TRAILER FILLER
WHITE
RIVER
TOILET
CAMPING AREA A
DAM SITE

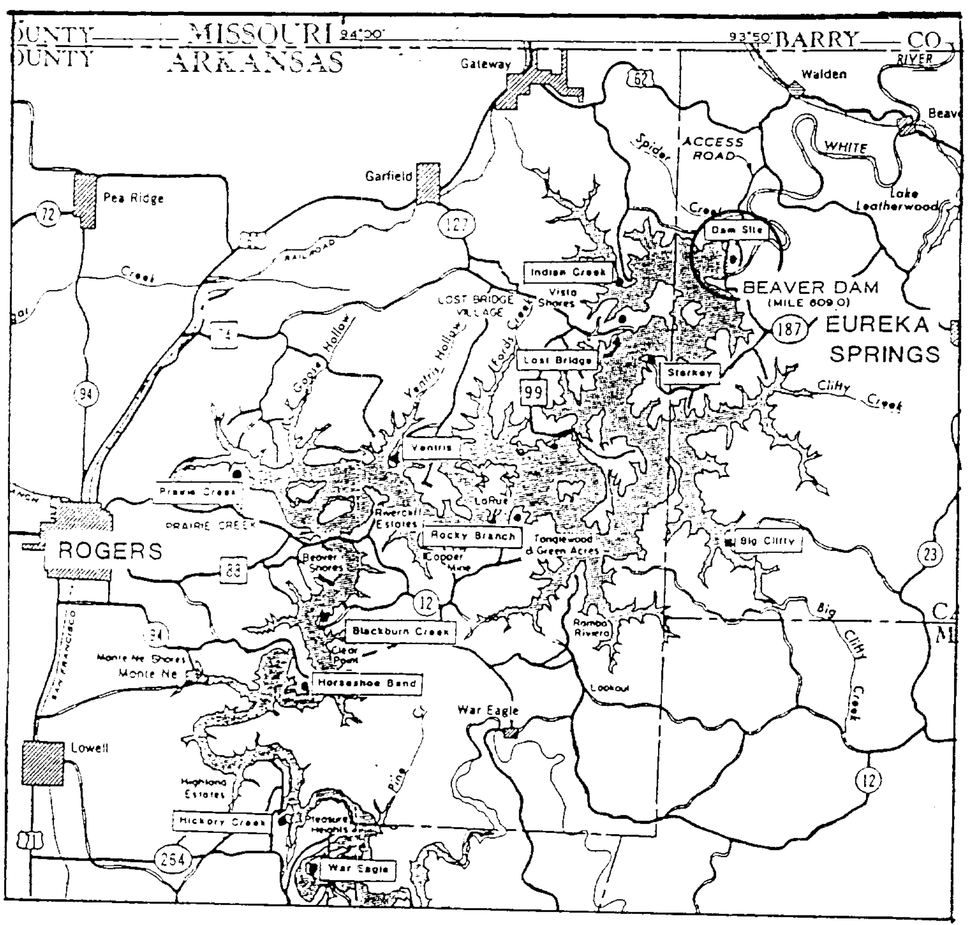

MISSOURI
ARKANSAS
BARRY
CO
Gateway
Walden
WHITE
RIVER
Lake
Leatherwood
ACCESS
ROAD
Spider
Creek
Garfield
Pea Ridge
72
127
62
Dam Site
Indian Creek
Vista
Shores
BEAVER DAM
(MILE 609.0)
187
EUREKA
SPRINGS
LOST BRIDGE
VILLAGE
Lost Bridge
99
Starkey
Clifty
Creek
94
Ventris
Prairie Creek
PRAIRIE CREEK
LaRue
Rocky Branch
Copper
Mine
Tanglewood
& Green Acres
Big Clifty
ROGERS
Beaver
Shores
88
12
Blackburn Creek
Clear
Point
Rambo
Rivera
Monte Ne Shores
Monte Ne
Horseshoe Bend
War Eagle
Lookout
Lowell
Highland
Estates
Hickory Creek
Treasure
Heights
254
War Eagle
71
23
12
Big
Clifty
Creek

PLATE A-1

WORK SAFELY

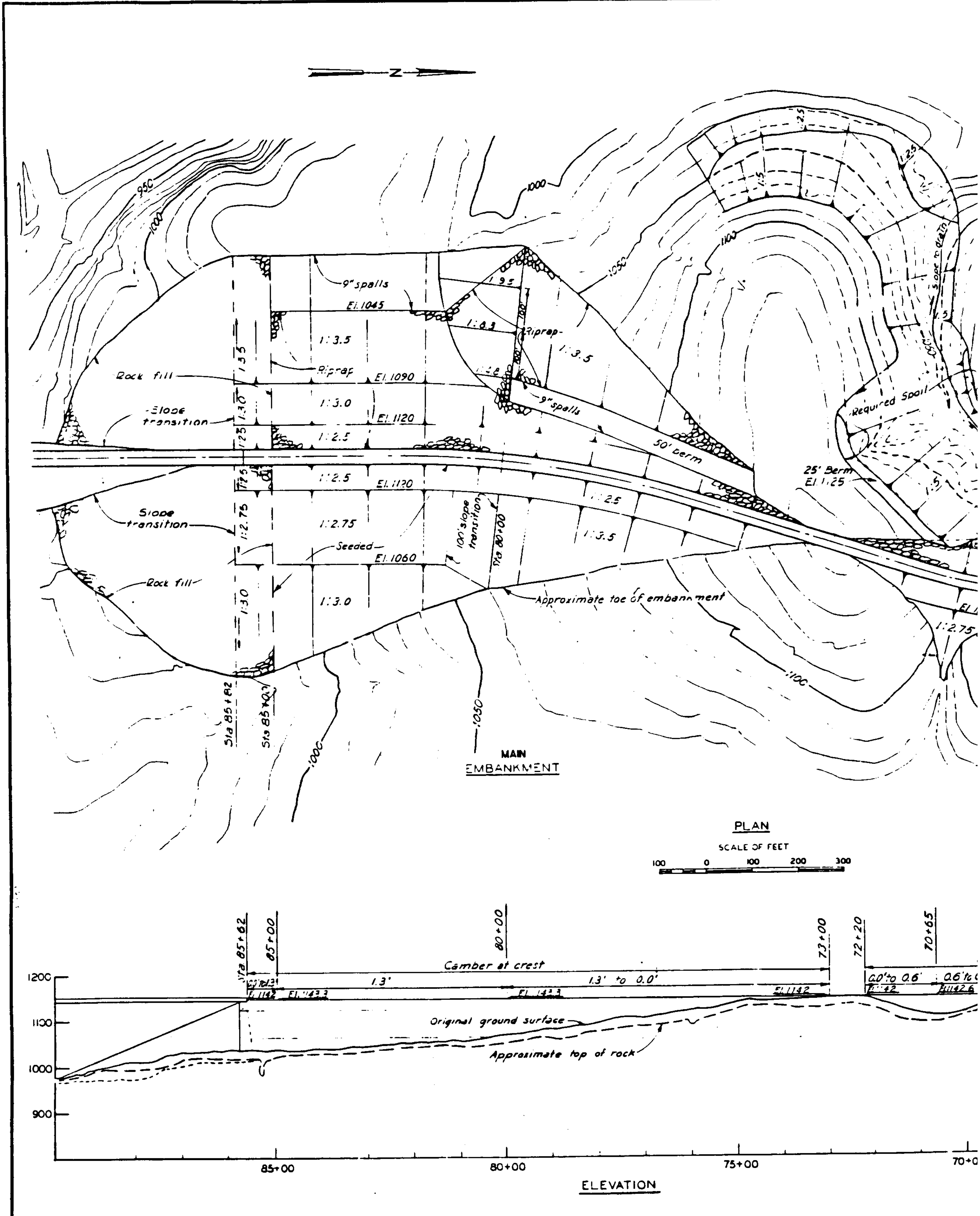
NOTES.

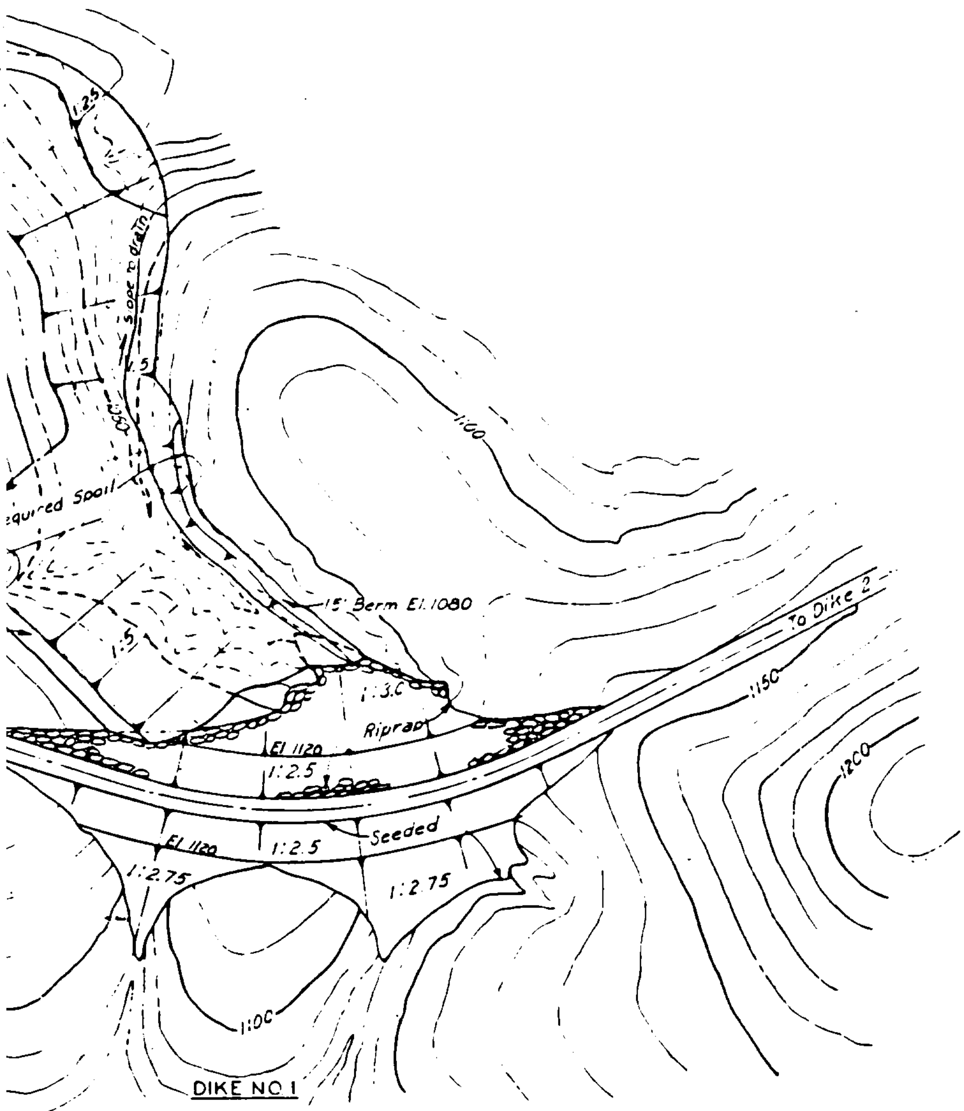
1. Borrow material to be supplied from Contractor source.
2. Contractor shall furnish slurry waste disposal site.
3. Rip-Rap, sand, gravel and aggregate sources are to be furnished by the Contractor.

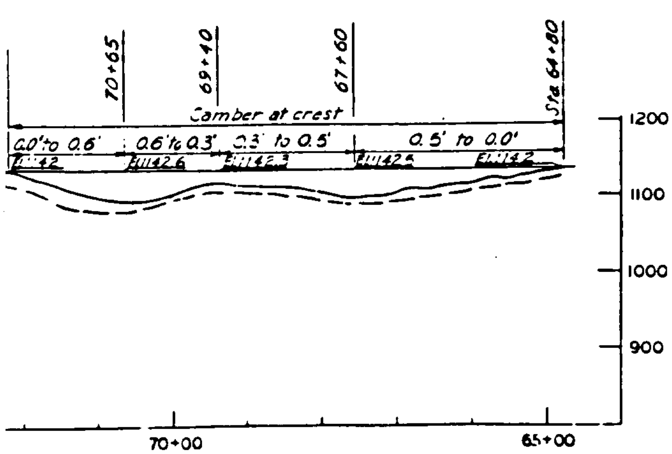
DEPARTMENT OF THE ARMY
LITTLE ROCK DISTRICT, CORPS OF ENGINEERS
LITTLE ROCK, ARKANSAS

DESIGNED BY: JH	WHITE RIVER — ARKANSAS
DRAWN BY: JKP	MAJOR REHABILITATION PROJECT CONCRETE CUTOFF WALL
CHECKED BY: JD	SITE PLAN BEAVER DAM – DIKE I
SUBMITTED BY:	SCALE: 1" = 200' (APPROX.) — OCTOBER 1991 — INV. SERIAL NO. 92 R-0191
ENGINEER	DRAWING NUMBER 10112-147/ 3219

PLATE A-2


N
9" spalls
El. 1045
1:3.5
Riprap
Rock fill
El. 1090
1:3.0
Slope transition
El. 1120
1:2.5
50' berm
El. 1130
1:2.75
Seeded
El. 1060
100' slope transition
Sta 80+00
1:3.5
Approximate toe of embankment
Rock fill
Slope transition
1:3.0
Sta 85+62
Sta 85+00
Required Spoil
25' Berm El. 1125
Slope to drain
MAIN
EMBANKMENT
PLAN
SCALE OF FEET
100 0 100 200 300
Camber at crest
1.3'
1.3' to 0.0'
0.0' to 0.6'
Original ground surface
Approximate top of rock
85+62
85+00
80+00
73+00
72+20
70+65
1200
1100
1000
900
85+00
80+00
75+00
70+00
ELEVATION





NOTE: CONTOURS FOR CONCEPT ONLY, NOT TO BE USED TO DETERMINE QUANTITIES.

REVISION	DATE	DESCRIPTION

DEPARTMENT OF THE ARMY
LITTLE ROCK DISTRICT, CORPS OF ENGINEERS
LITTLE ROCK, ARKANSAS

DESIGNED BY: JAB
DRAWN BY: JAB
CHECKED BY: RLA

WHITE RIVER — ARKANSAS

MAJOR REHABILITATION PROJECT
CONCRETE CUTOFF WALL

MAIN EMBANKMENT AND DIKE - 1
PLAN AND ELEVATION
BEAVER DAM - DIKE 1

SUBMITTED BY: Carroll D. Winter
ENGINEER

SCALE: AS SHOWN

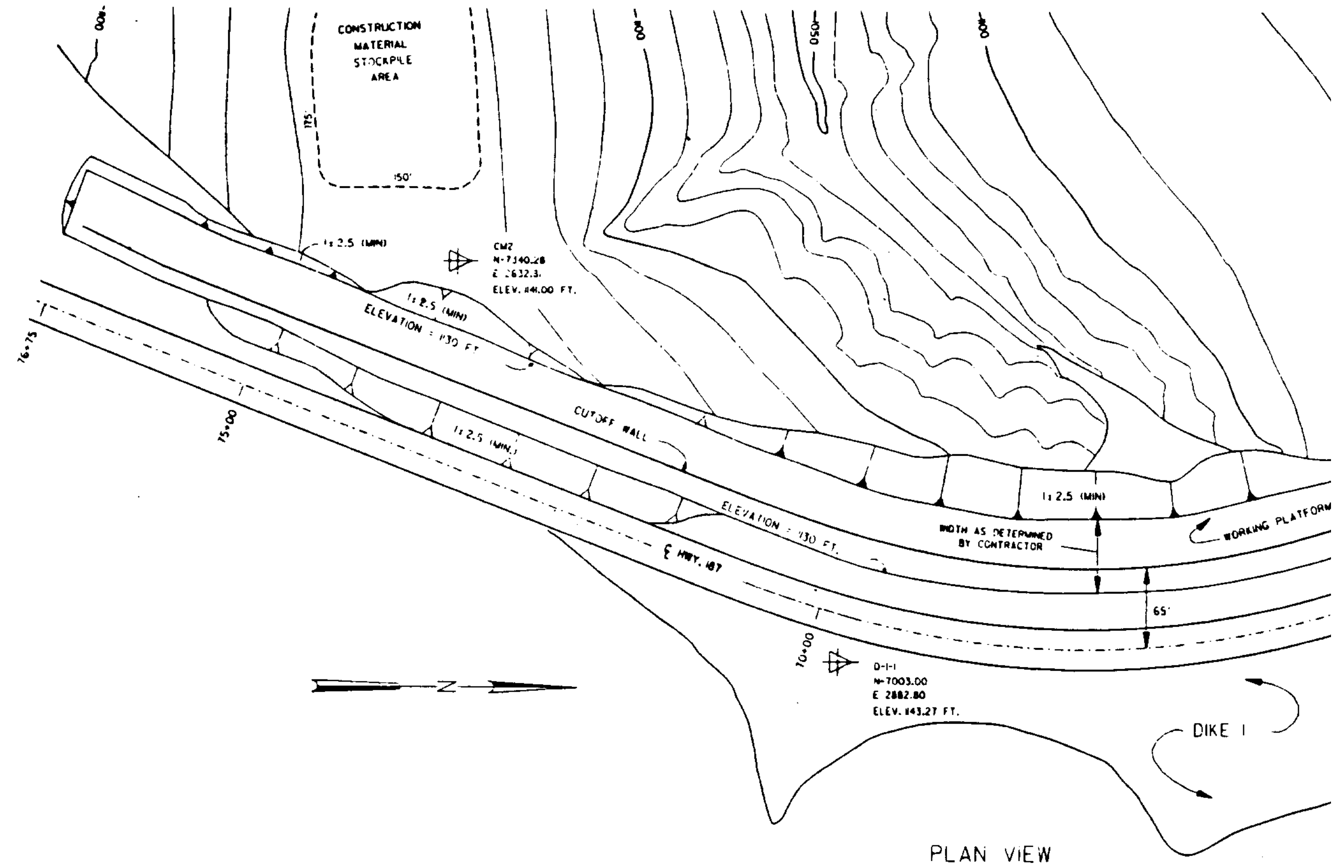
OCTOBER 1991

INV. SERIAL NO.: 92-R-0001

DRAWING NUMBER: 10112-147/13220

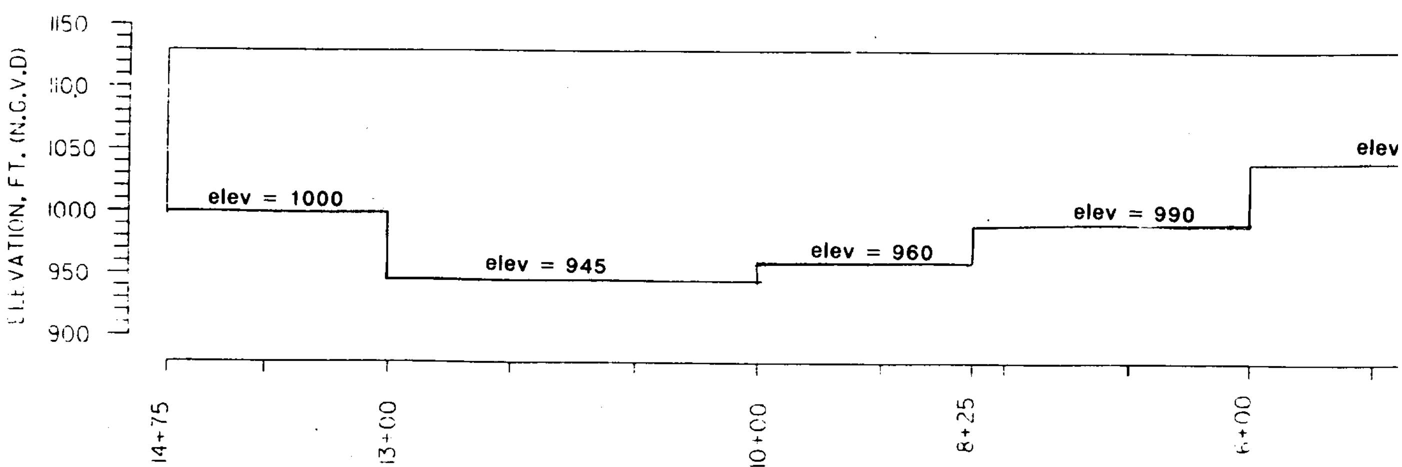
SHEET 2 OF 16

PLATE A-3



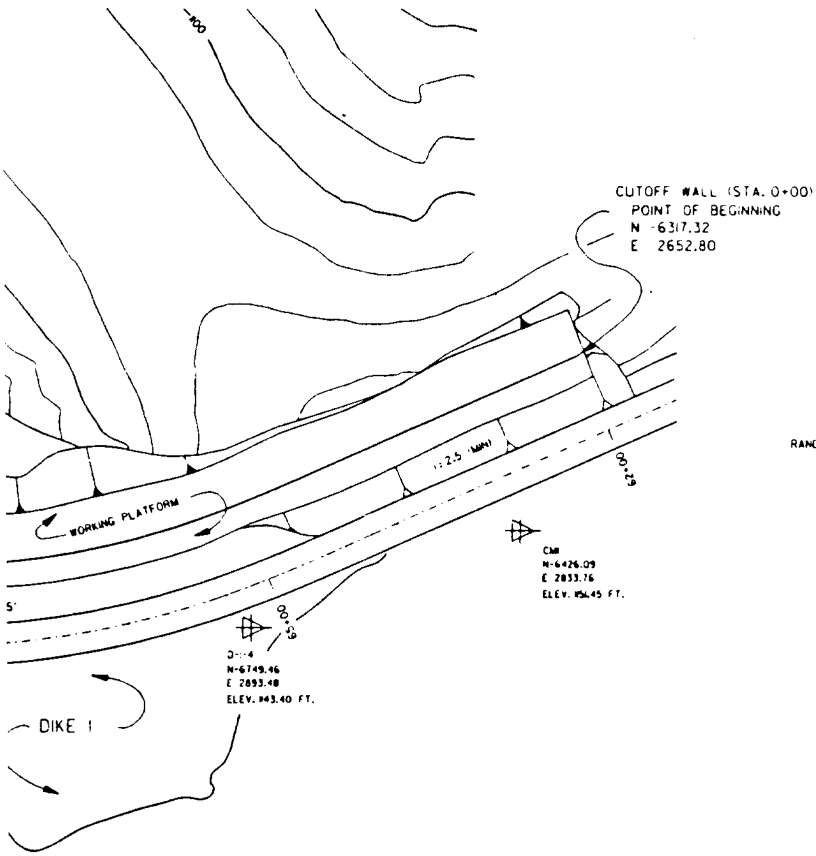
PLAN VIEW

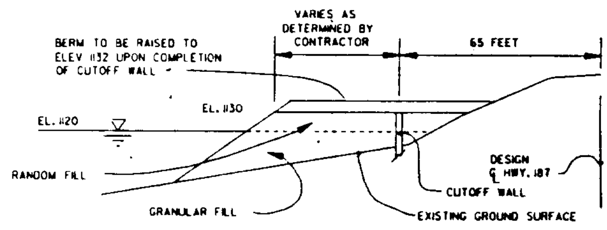
0 60 120
SCALE IN FEET



PROFILE ALONG CENTER LINE OF WALL

0 60 120
SCALE IN FEET

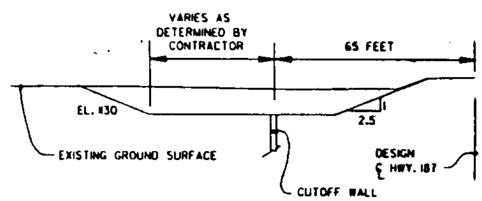




TYPICAL FILL SECTION

STA. 8+00

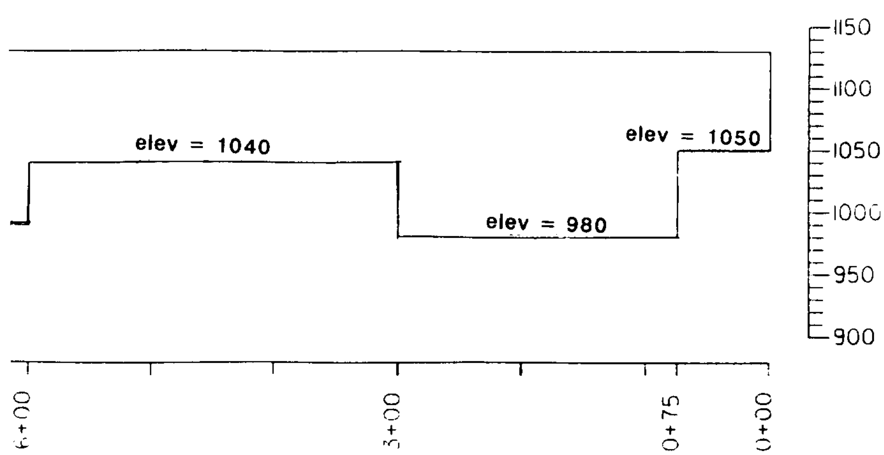
(N.T.S.)



TYPICAL CUT SECTION

STA. 12+00

(N.T.S.)



NOTES:

1. CONTOURS FOR INFORMATION ONLY, NOT TO BE USED TO DETERMINE QUANTITIES.
2. CUT SECTIONS OF THE WORK PLATFORM ACROSS THE ROADS OF DAY USE PARK AREAS NO. 3 AND 4 SHALL BE BACKFILLED TO RESTORE ACCESS TO THESE TO AN EQUAL OR BETTER CONDITION THAN THEY WERE PRIOR TO CONSTRUCTION.

REL.	DESCRIPTION

DEPARTMENT OF THE ARMY
LITTLE ROCK DISTRICT, CORPS OF ENGINEERS
LITTLE ROCK, ARKANSAS

DESIGNED BY: JAB
DRAWN BY: JAB
CHECKED BY: RLA

WHITE RIVER — ARKANSAS

MAJOR REHABILITATION PROJECT
CONCRETE CUTOFF WALL

CUTOFF WALL - PLAN AND PROFILE
BEAVER DAM - DIKE I

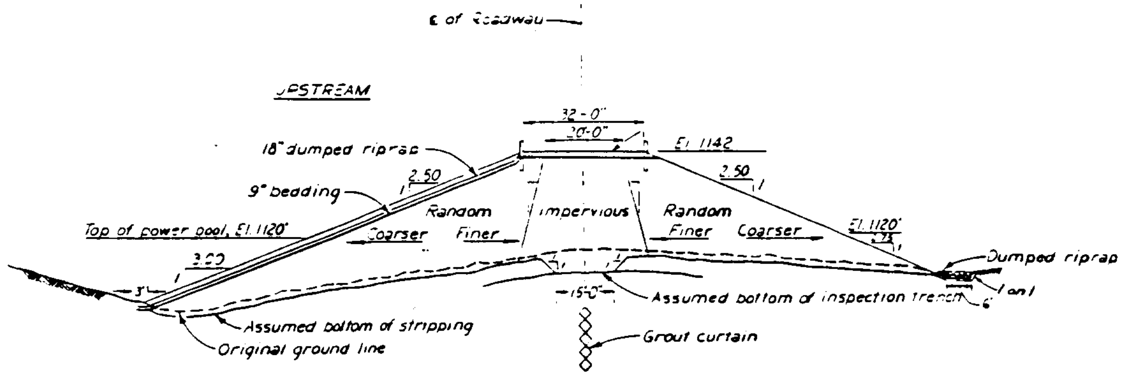
SUBMITTED BY: Carroll D. Winter
ENGINEER

OCTOBER 1991

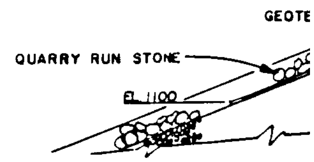
92-R-0001

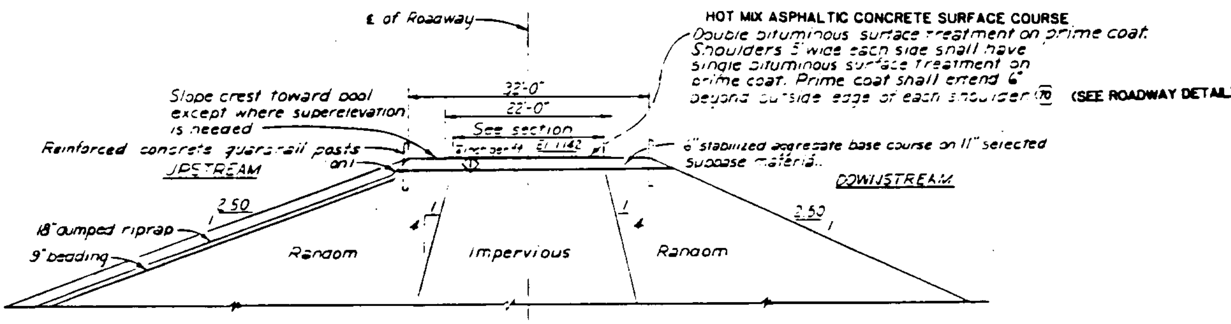
10112-147/3221

PLATE A-4

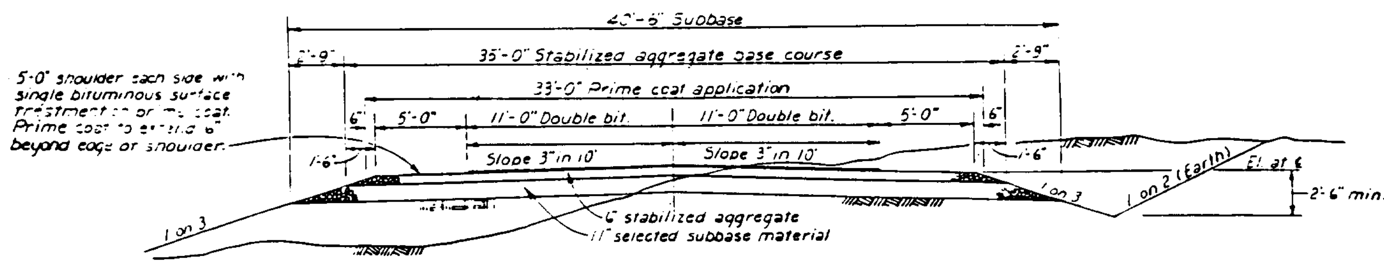


DIKE 1
STATION 68+00
SCALE OF FEET
20 0 20

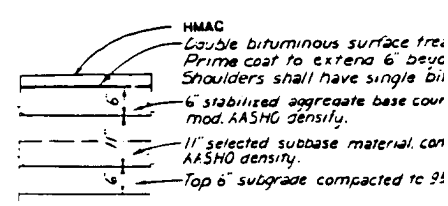




DIKE 1 AND DIKE 2
CREST DETAIL
SCALE OF FEET
10 0 10

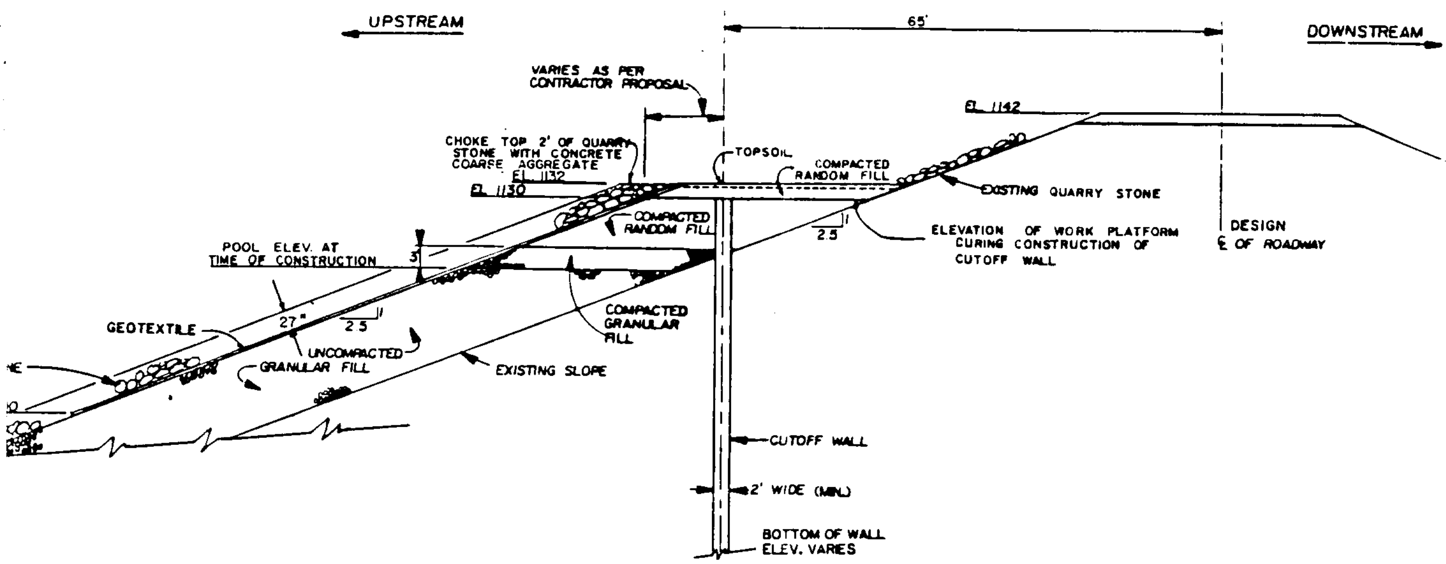


SECTION THRU ROAD TO DAM
SCALE OF FEET
0 5



ROADWAY DETAIL

6 7 8 9 10



CUTOFF WALL
TYPICAL SECTION

10 5 0 10
SCALE IN FEET

G
E
D
C
B

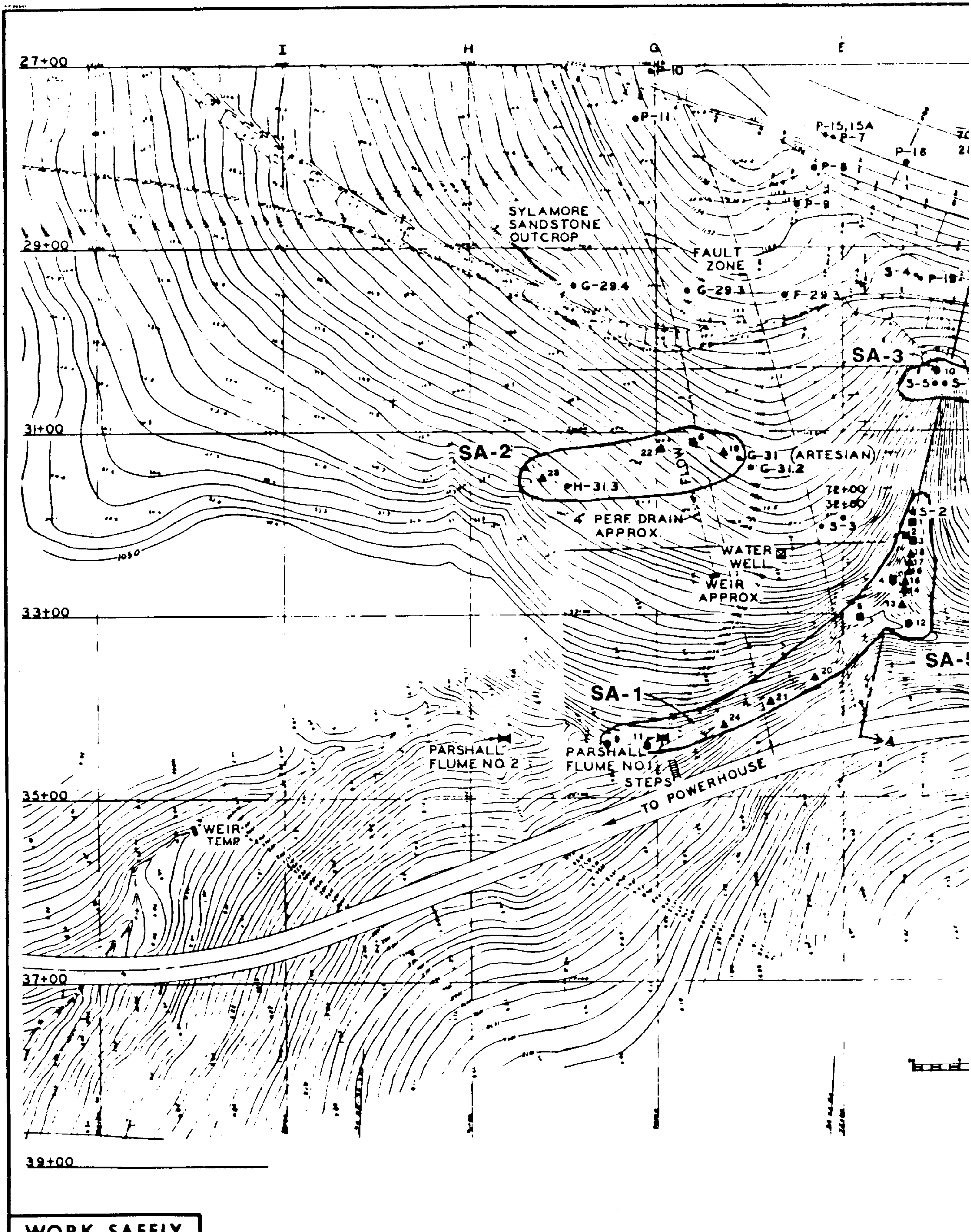
" min.

ous surface treatment on prime coat.
extend 6" beyond outside of each shoulder.
have single bituminous surface treatment.
regate base course, compacted to 100%
sity.
ase material, compacted to 95% mod
compacted to 95% mod. AASHO density.

REVISION	DATE	DESCRIPTION

DEPARTMENT OF THE ARMY
LITTLE ROCK DISTRICT, CORPS OF ENGINEERS
LITTLE ROCK ARKANSAS

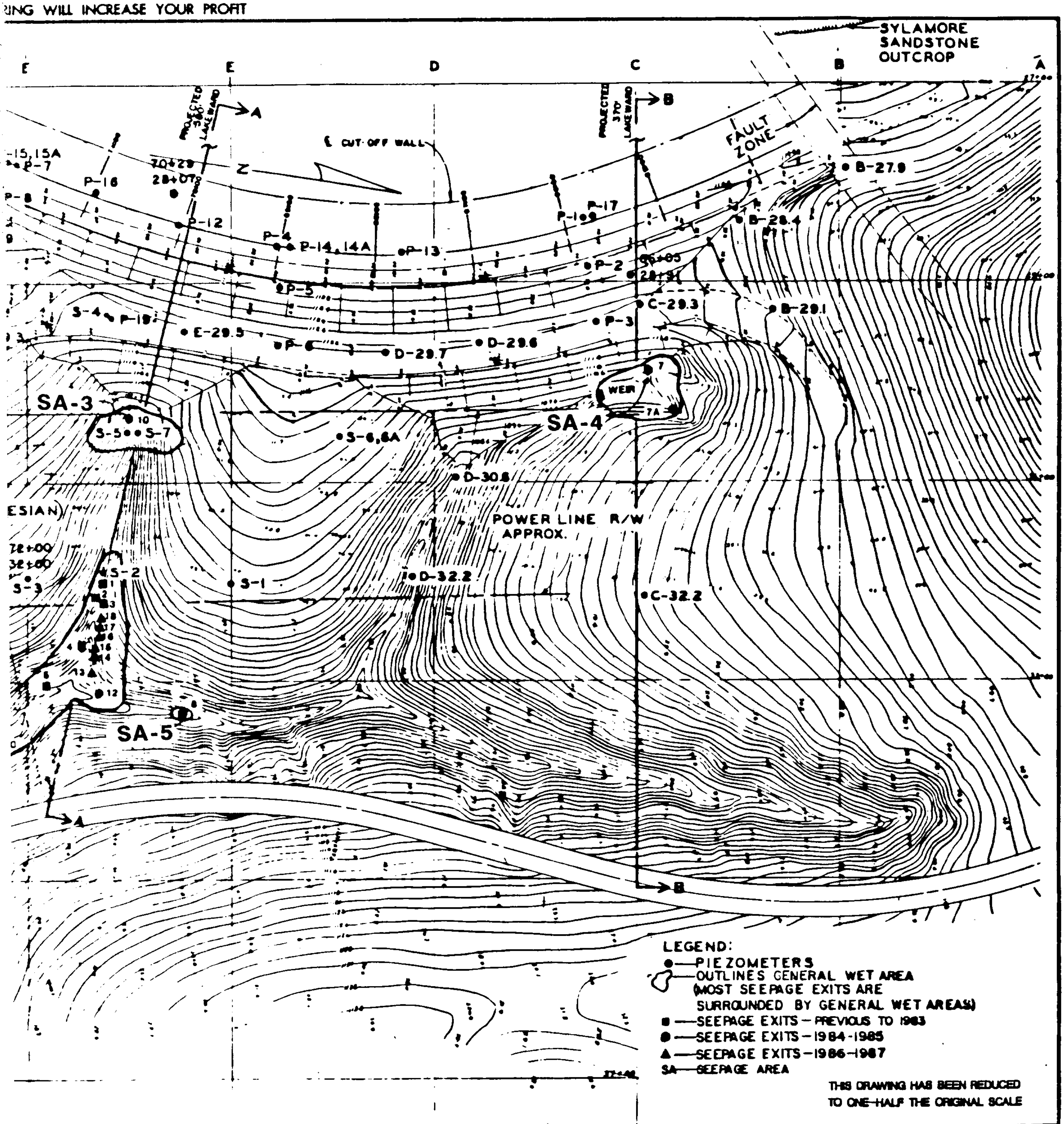
DESIGNED BY: BRA	WHITE RIVER — ARKANSAS MAJOR REHABILITATION PROJECT CONCRETE CUTOFF WALL
DRAWN BY: JTD	TYPICAL SECTIONS
CHECKED BY: RLA	BEAVER DAM – DIKE I
SUBMITTED BY: Carroll E. Winter, ENGINEER	SCALE: AS SHOWN — OCTOBER 1991 — INV. SERIAL NO. 92-R-0001
	SHEET 12 OF 16 — DRAWING NUMBER 10112-147/3229



I
H
G
E
27+00
29+00
31+00
33+00
35+00
37+00
39+00
P-10
P-11
P-15,15A
P-7
P-16
P-8
P-9
SYLAMORE
SANDSTONE
OUTCROP
FAULT
ZONE
S-4
G-29.4
G-29.3
F-29.3
SA-3
S-5
SA-2
PH-31.3
FLOW
G-31 (ARTESIAN)
G-31.2
4" PERF. DRAIN
APPROX.
S-3
S-2
WATER
WELL
WEIR
APPROX.
1050
SA-1
PARSHALL
FLUME NO 2
PARSHALL
FLUME NO.1
STEPS
TO POWERHOUSE
WEIR
TEMP

WORK SAFELY

ING WILL INCREASE YOUR PROFIT



LEGEND:

- ● — PIEZOMETERS
- — OUTLINES GENERAL WET AREA (MOST SEEPAGE EXITS ARE SURROUNDED BY GENERAL WET AREAS)
- ■ — SEEPAGE EXITS – PREVIOUS TO 1983
- ● — SEEPAGE EXITS – 1984-1985
- ▲ — SEEPAGE EXITS – 1986-1987
- SA — SEEPAGE AREA

THIS DRAWING HAS BEEN REDUCED TO ONE-HALF THE ORIGINAL SCALE

SCALE OF FEET

NOTES

SEE DRAWINGS 3227 AND 3228 FOR CROSS SECTIONS A-A AND B-B.

DEPARTMENT OF THE ARMY
LITTLE ROCK DISTRICT, CORPS OF ENGINEERS
LITTLE ROCK, ARKANSAS

WHITE RIVER — ARKANSAS

MAJOR REHABILITATION PROJECT
CONCRETE CUTOFF WALL

LOCATION OF PIEZOMETERS AND CROSS SECTIONS

BEAVER DAM – DIKE I

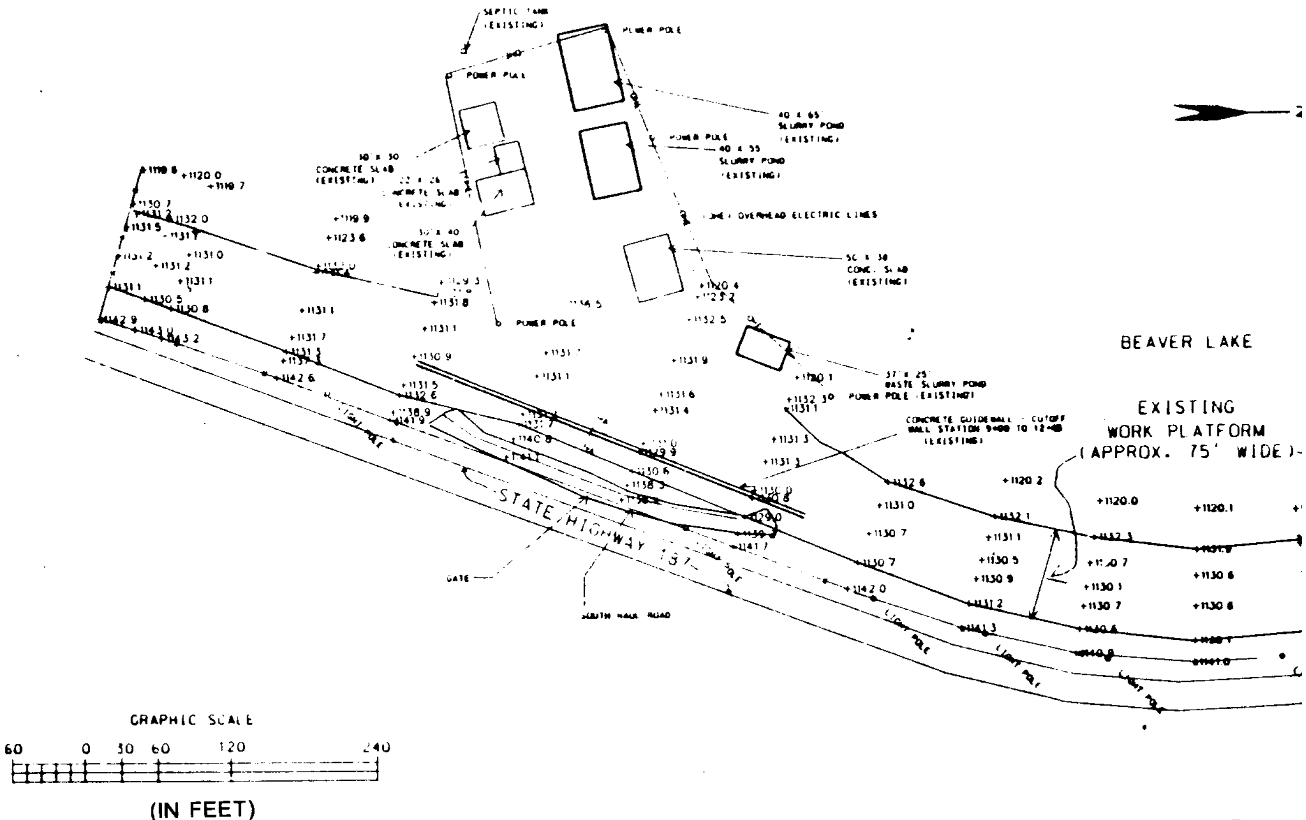
DESIGNED BY: BH
DRAWN BY: JTD
CHECKED BY: CD
SUBMITTED BY: Carroll D. Winter

SCALE: AS SHOWN

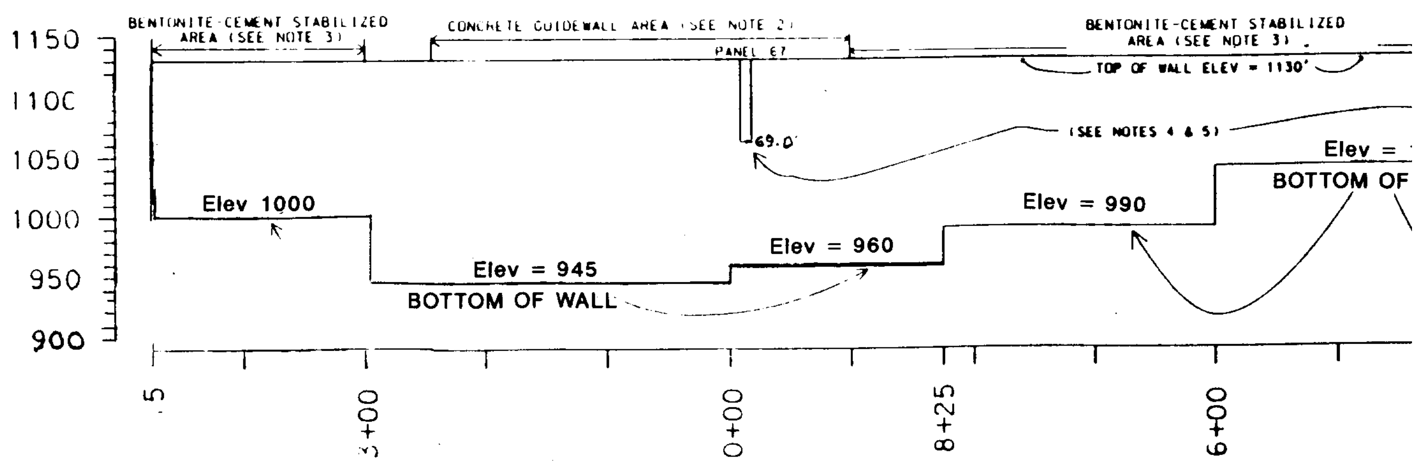
OCTOBER 1991

10112-147/ 3226

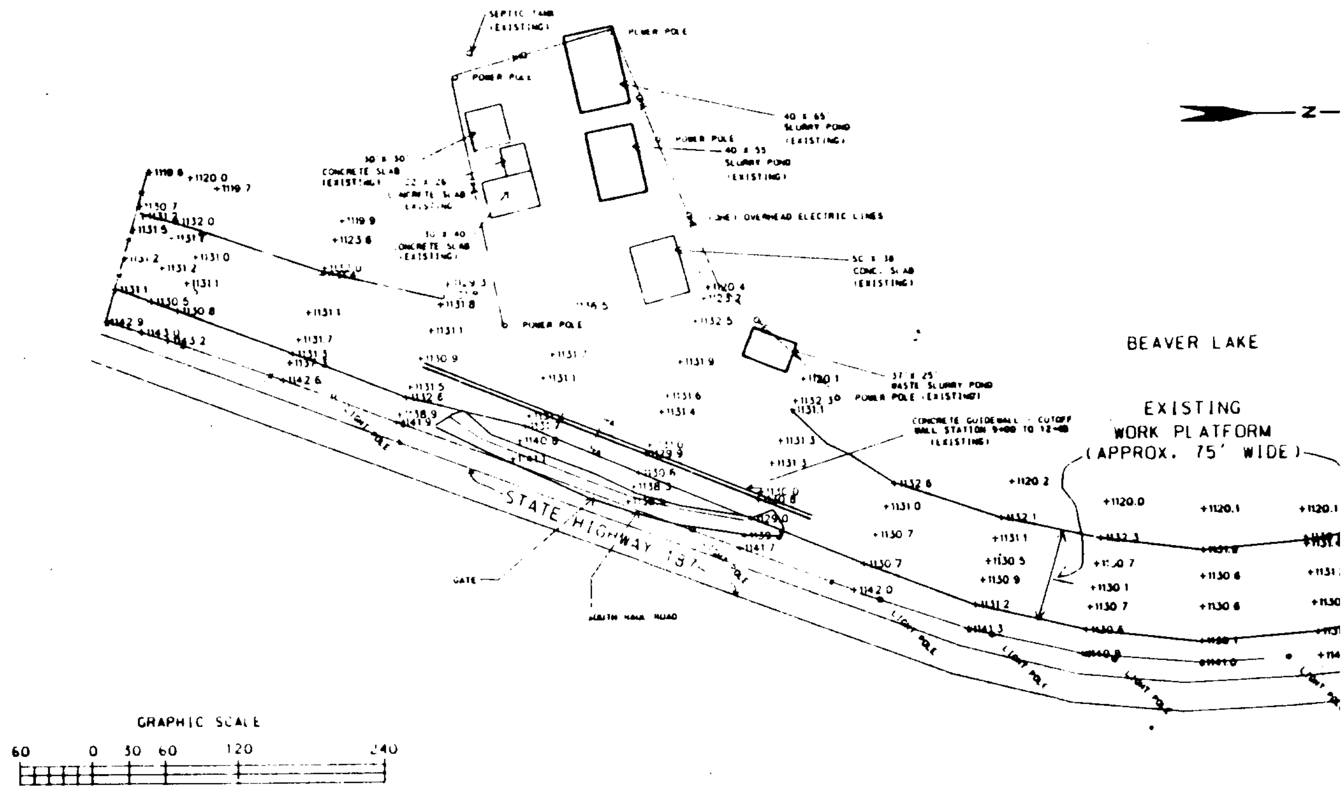
PLATE A-6



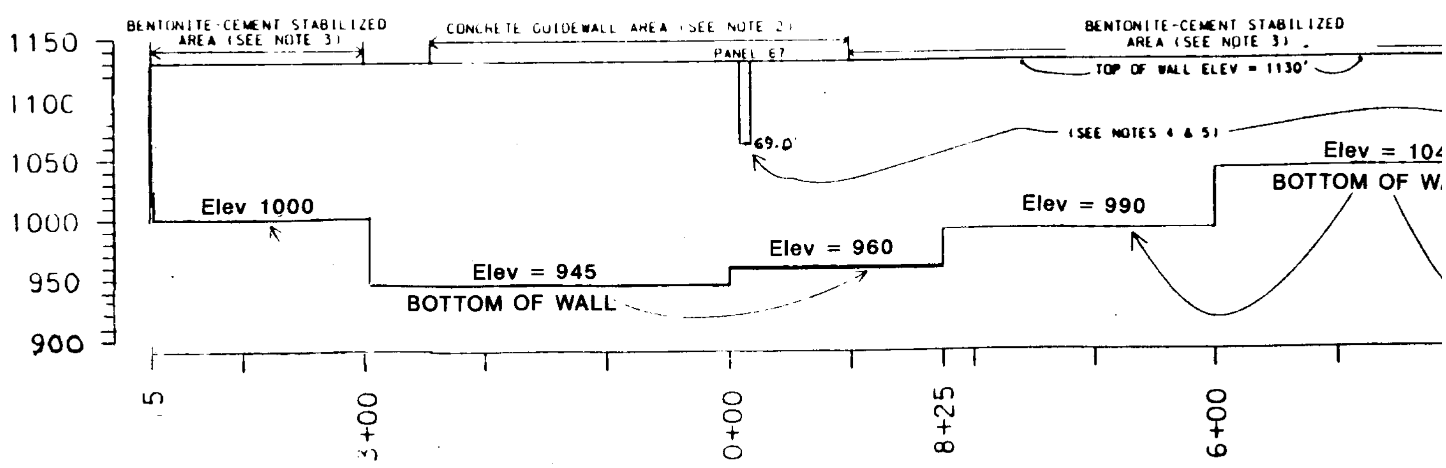
EXISTING WORK PLATFORM - PLAN



PROFILE ALONG ℄ OF CUTOFF WAL
WITH EXCAVATED PANELS

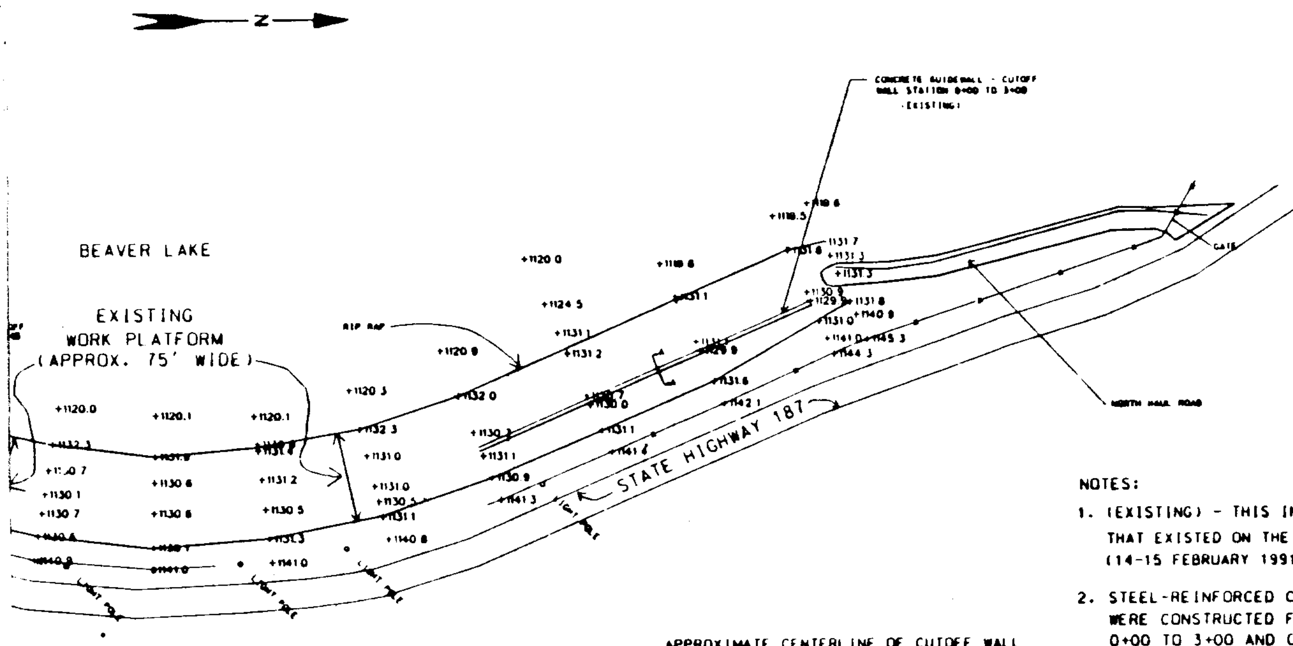


EXISTING WORK PLATFORM - PLAN V

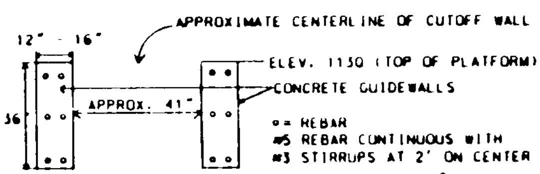


PROFILE ALONG ℄ OF CUTOFF WALL
WITH EXCAVATED PANELS

WORK SAFELY



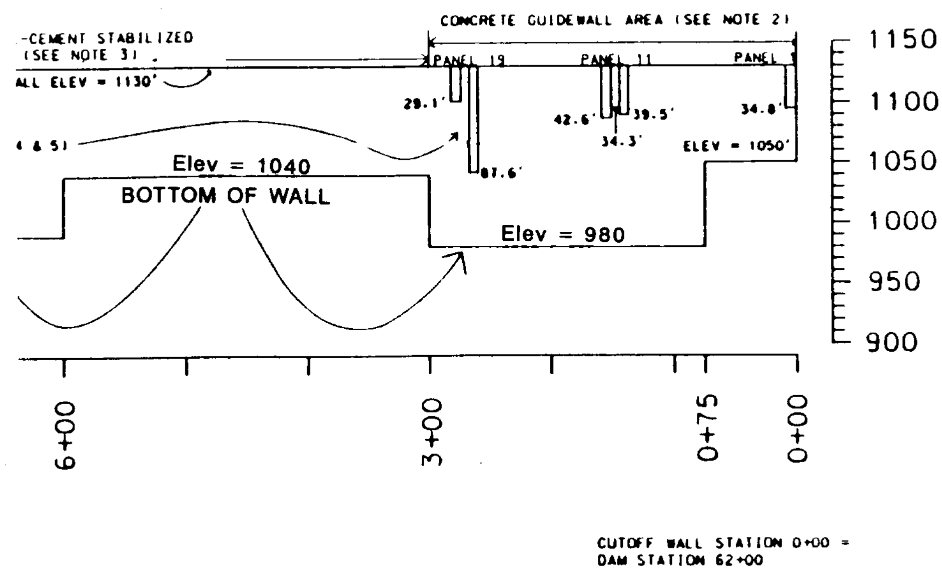
.ATFORM - PLAN VIEW



GUIDEWALL DETAIL
N.T.S.
SECTION A-A
(SEE NOTE 2)

NOTES:

1. (EXISTING) - THIS INDICATES AN ITEM AND/OR AREA THAT EXISTED ON THE DATE OF THE SURVEY (14-15 FEBRUARY 1991) FROM A PREVIOUS CONTRACT.
2. STEEL-REINFORCED CONCRETE GUIDEWALLS WERE CONSTRUCTED FROM CUTOFF WALL STATIONS 0+00 TO 3+00 AND CUTOFF WALL STATIONS 9+00 TO 12+45. UNDER A PREVIOUS CONTRACT. THE TOP OF THE GUIDEWALLS ARE AT ELEVATION 1130. THEY ARE APPROXIMATELY 36" DEEP. 12"-16" WIDE AND 41" APART. (SEE SEC. A-A)
3. BENTONITE/CEMENT STABILIZATION WAS PERFORMED BETWEEN CUTOFF WALL STATIONS 3+00 - 9+00 AND CUTOFF WALL STATIONS 13+00 - 14+75. UNDER A PREVIOUS CONTRACT. THE STABILIZED AREA IS APPROXIMATELY 6' WIDE AND VARIES FROM 4'-10' IN DEPTH.
4. EXCAVATION IDENTIFICATION NUMBERS (PANELS 1,11, 19 AND 67) ARE FROM PREVIOUS CONTRACT. AND ARE FOR INFORMATION ONLY. THESE EXCAVATIONS WERE BACKFILLED WITH CUTTINGS.
5. EXCAVATED PANELS ARE APPROXIMATELY 9' LONG (PARALLEL TO THE CENTERLINE OF THE WALL) AND 3' WIDE. DEPTHS OF EACH PANEL VARY AS SHOWN ON THE PROFILE.



CUTOFF WALL STATION 0+00 =
DAM STATION 62+00
65' UPSTREAM OF HIGHWAY DESIGN CENTERLINE

OF CUTOFF WALL
TED PANELS

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
LITTLE ROCK DISTRICT, CORPS OF ENGINEERS
LITTLE ROCK, ARKANSAS

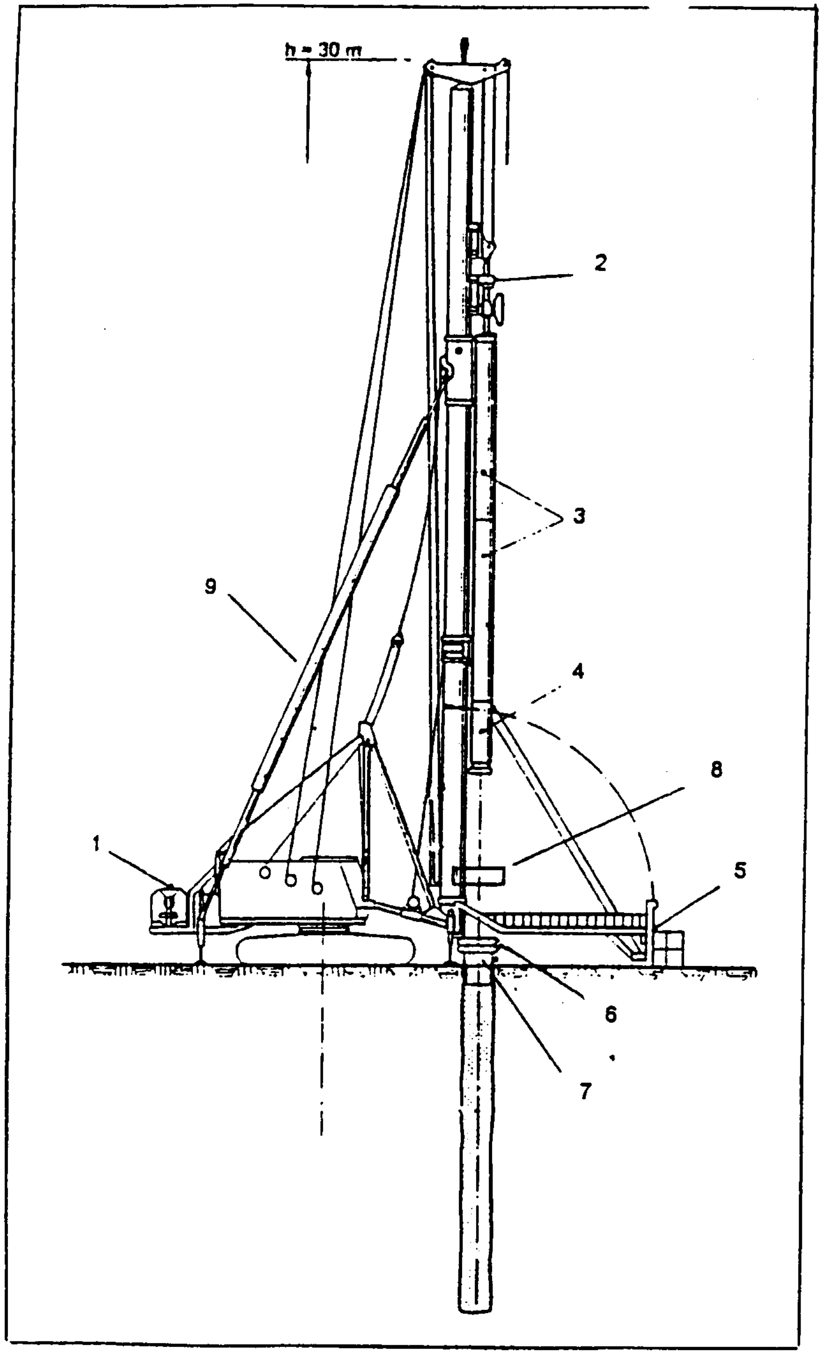
DESIGNED BY: JB
DRAWN BY: SC
CHECKED BY: HH

WHITE RIVER — ARKANSAS
MAJOR REHABILITATION PROJECT
CONCRETE CUTOFF WALL
WORK PLATFORM - PLAN VIEW
(EXISTING)
BEAVER DAM - DIKE 1

SUBMITTED BY: Carroll D. Winter
ENGINEER

OCTOBER 1991
INV. SERIAL NO. 92-R-0001
DRAWING NUMBER 10112-147/3231
SHEET 14 OF 18

PLATE A-7



RODIO NICHOLSON JOINT VENTURE
BEAVER DAM PROJECT

SCHEME OF DRILL RIG

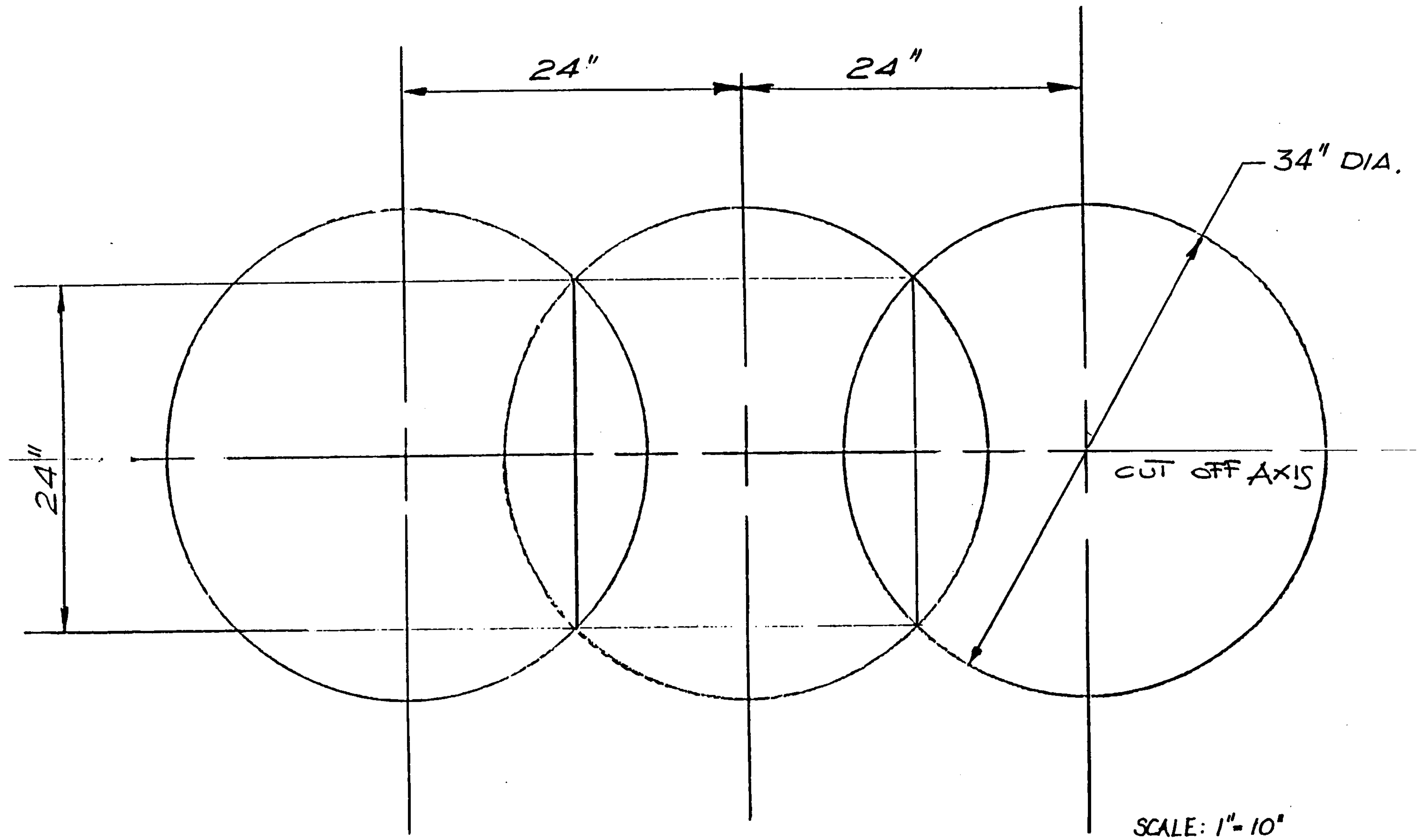
LEGEND

1. Hydraulic power pack, diesel driven
2. Rotary head (hydraulic power swivel)
3. Drill rods, 30 feet long x 32 inches O.D.
4. Down-the-hole hammer
5. Drill rod loader
6. Lower guide
7. Drill cuttings exhauster
8. Drill rod breakout wrench
9. Tower bracing and verticality links

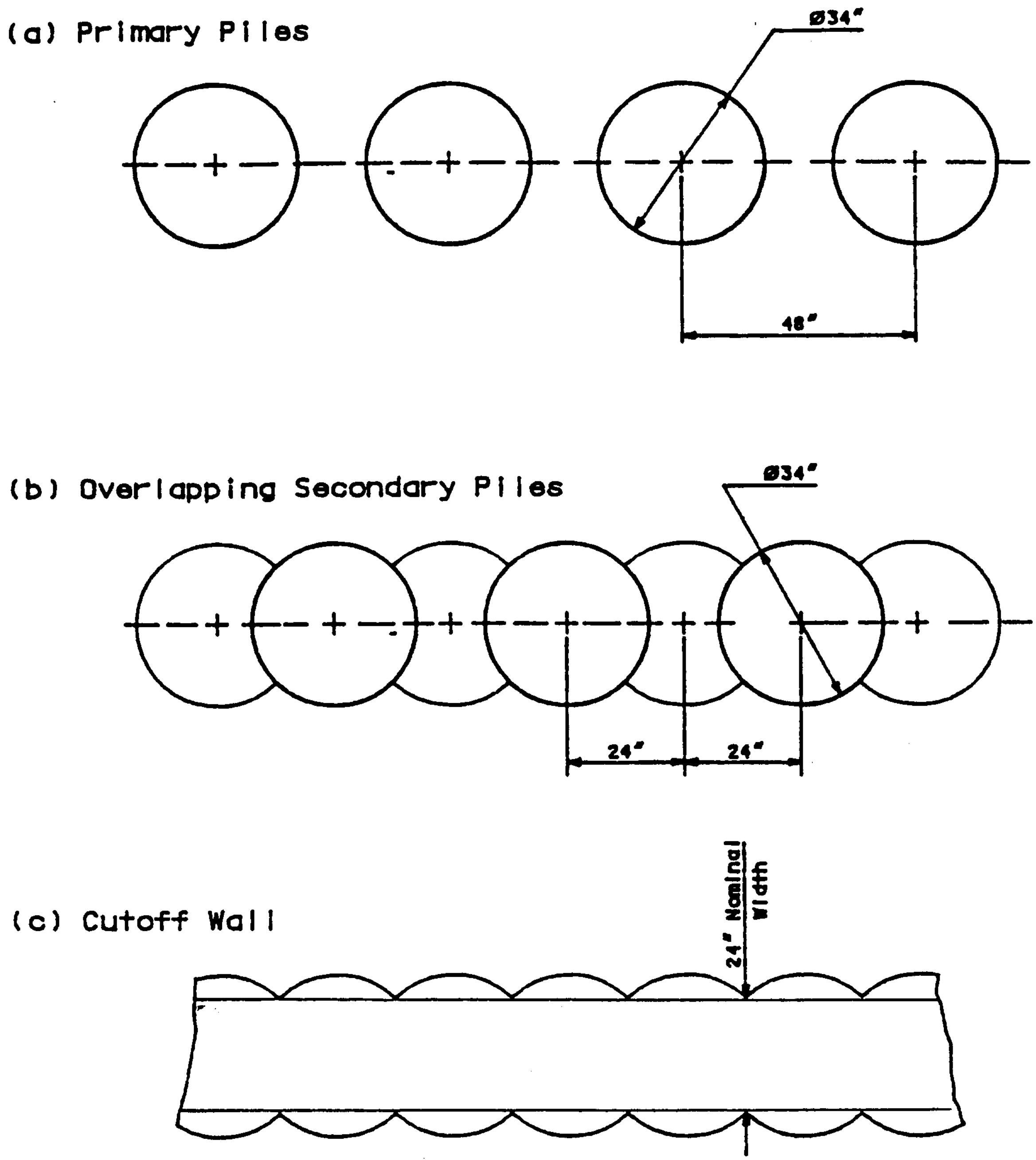
NOTES

** The drill rig shown is actually the one denominated "EC-80 SUPERDRILL" (rig no. 1)

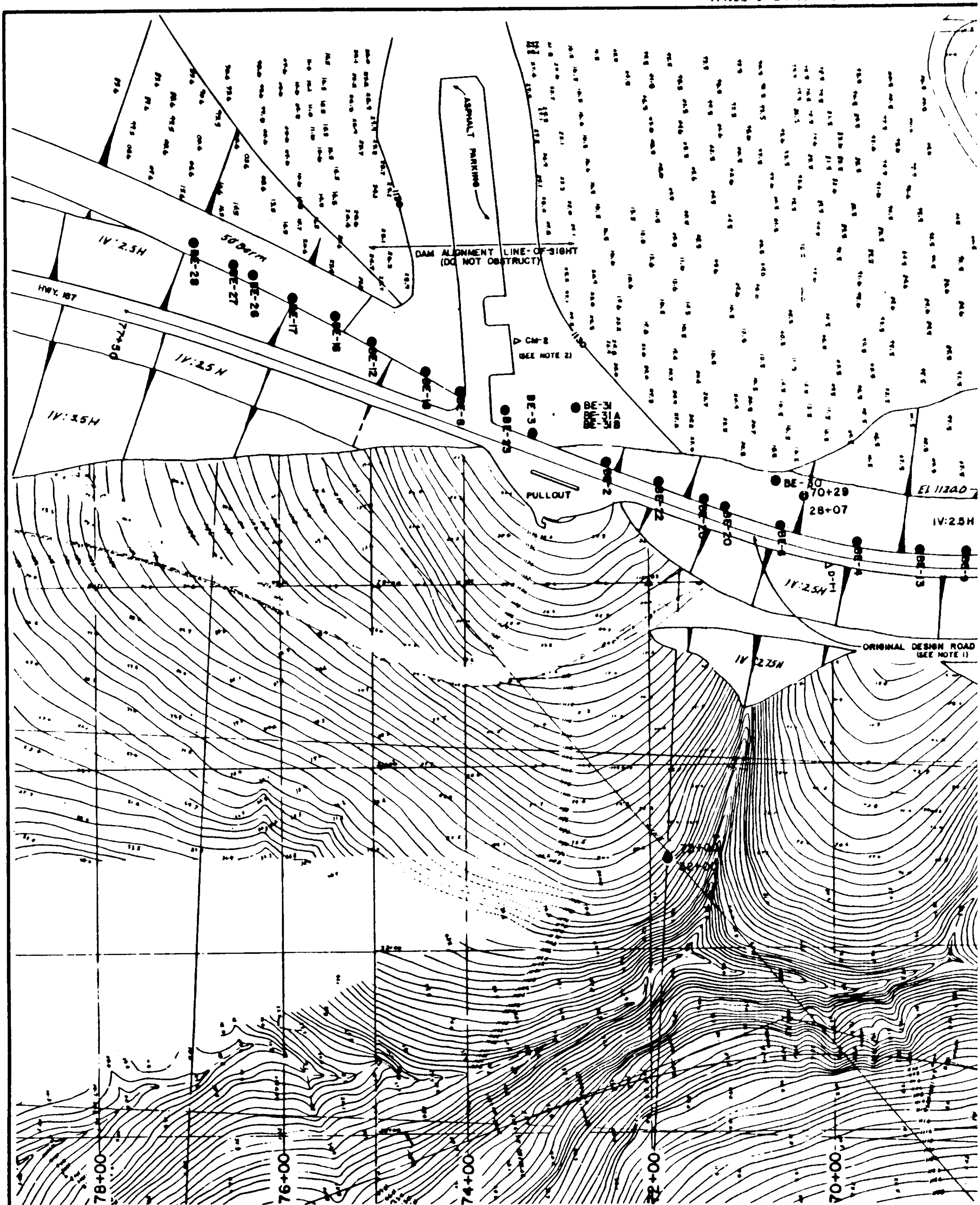
** The functional scheme of rig no. 2 is conceptually identical. However, the rig is equipped with two drill rod changers, instead of the drill rod loader. In this way, the drill rig can carry a full complement of 180 feet of drill rods



PILE LAYOUT AND PRINCIPAL DIMENSIONS - 34" DIA. PILES



Construction phases of the cutoff wall
(Single row, 34" dia. Secant Piles)


ASPHALT PARKING
DAM ALIGNMENT LINE-OF-SIGHT (DO NOT OBSTRUCT)
50' Berm
1V:2.5H
HWY. 187
77+50
1V:3.5H
CM-2 (SEE NOTE 2)
BE-31
BE-31A
BE-31B
PULLOUT
BE-30
70+29
28+07
1V:2.5H
ORIGINAL DESIGN ROAD (SEE NOTE 1)
1V:2.25H
78+00
76+00
74+00
72+00
70+00

WORK SAFELY

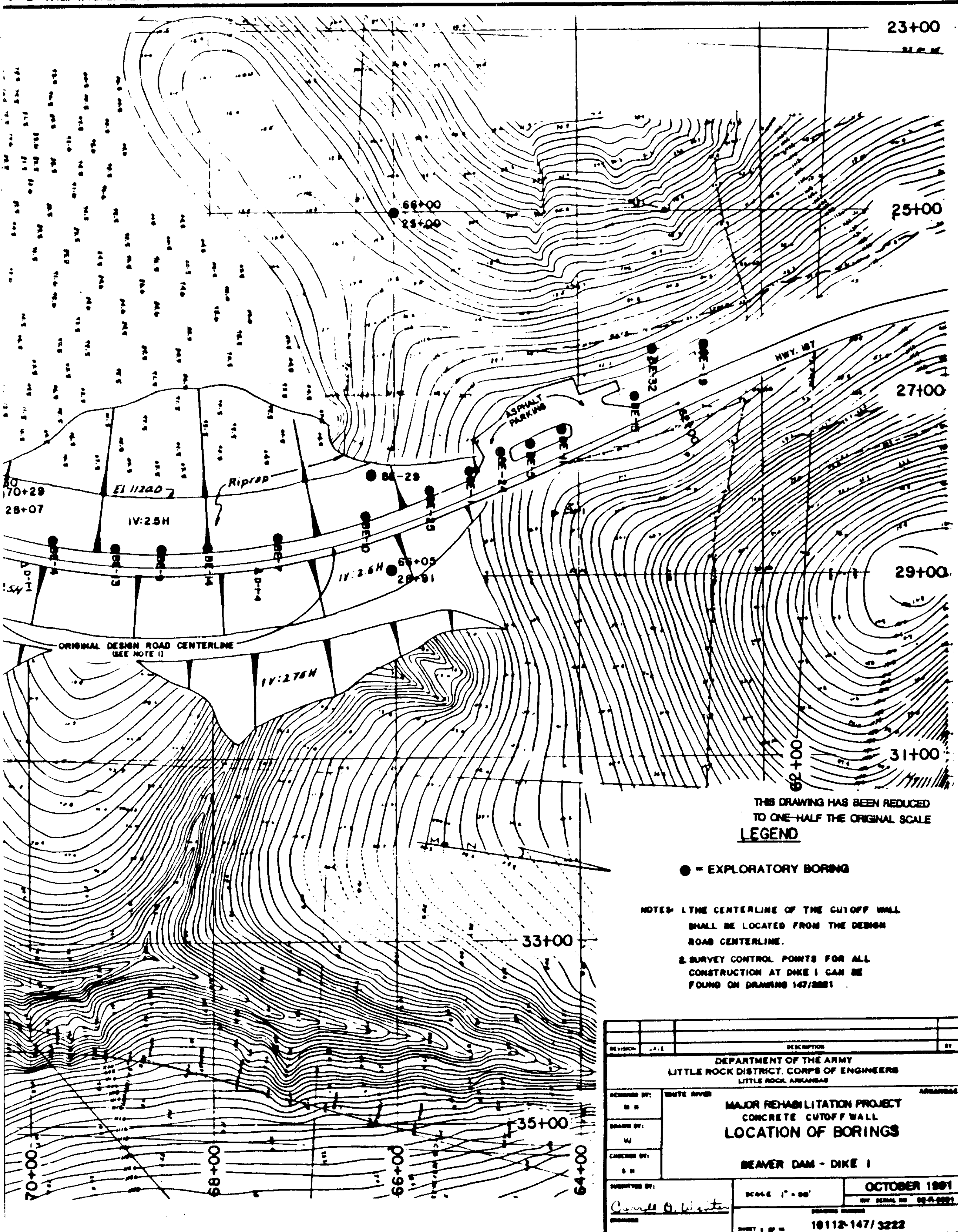
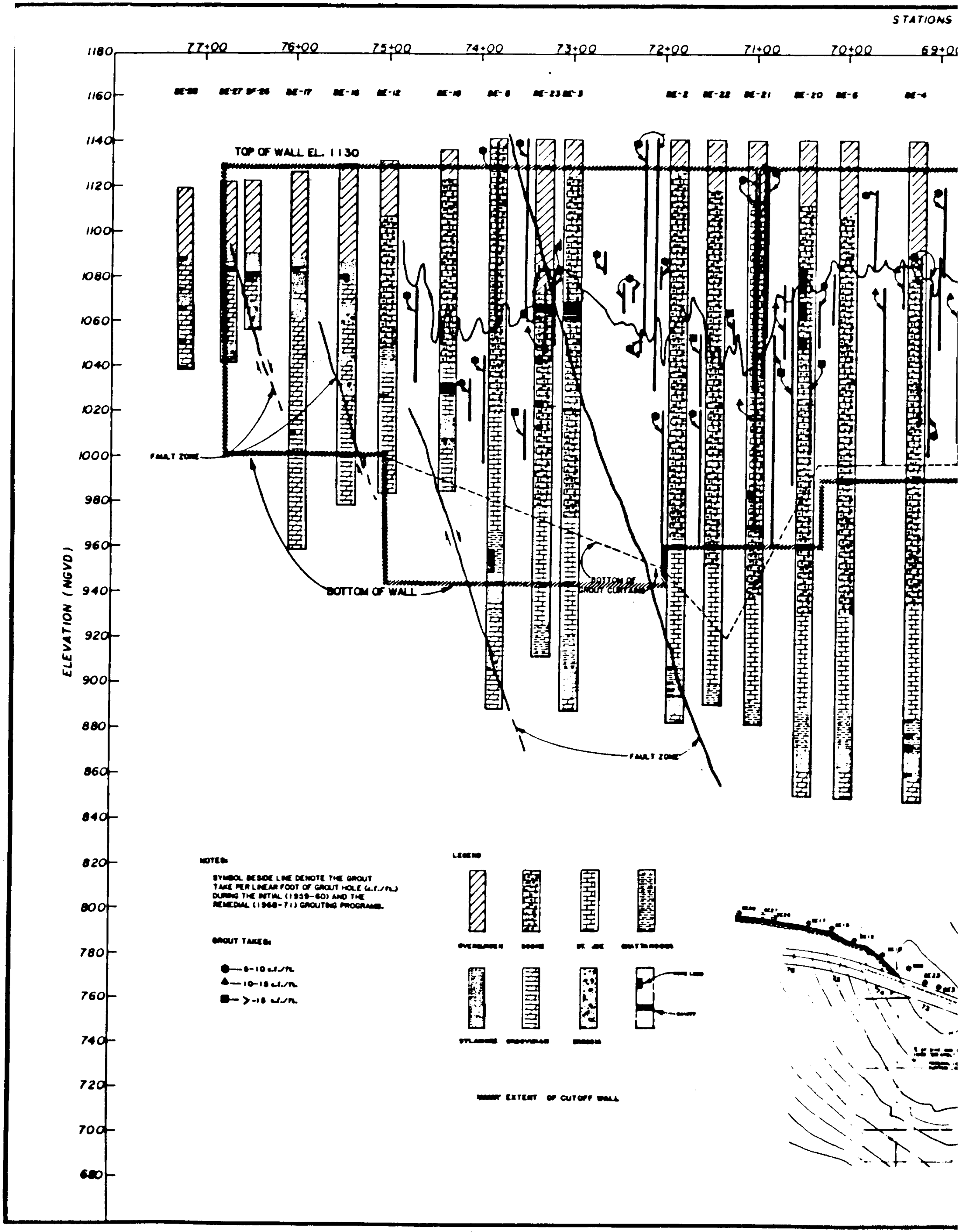
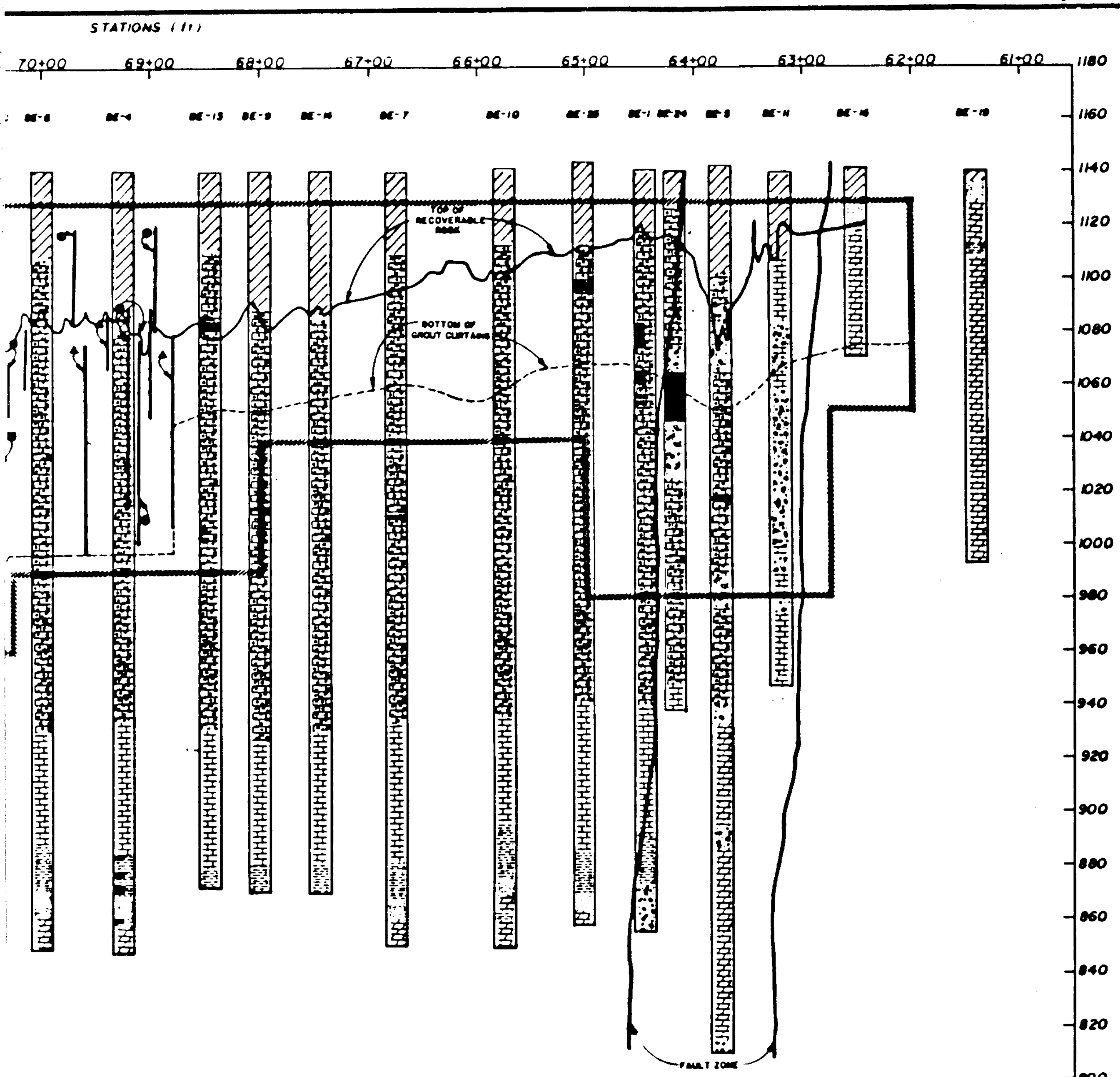
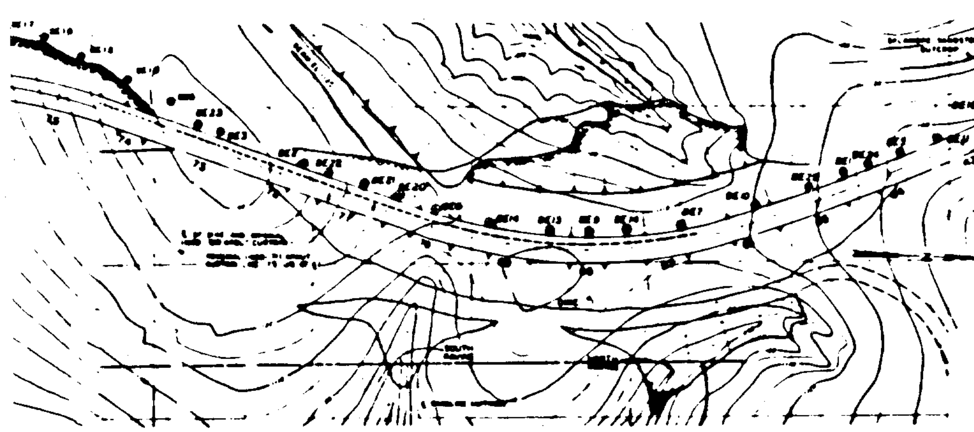

23+00
25+00
27+00
29+00
31+00
33+00
35+00
66+00
25+00
65+05
28+91
70+29
28+07
70+00
68+00
66+00
64+00
62+00
HWY. 187
ASPHALT PARKING
BE-32
BE-28
BE-25
BE-10
BE-13
BE-14
BE-7
AD-1
AD-4
El 1120.0
Riprap
1V:2.5H
1V:2.6H
1V:2.75H
ORIGINAL DESIGN ROAD CENTERLINE
(SEE NOTE 1)
THIS DRAWING HAS BEEN REDUCED
TO ONE-HALF THE ORIGINAL SCALE
LEGEND
● = EXPLORATORY BORING
NOTES: 1. THE CENTERLINE OF THE CUTOFF WALL SHALL BE LOCATED FROM THE DESIGN ROAD CENTERLINE.
2. SURVEY CONTROL POINTS FOR ALL CONSTRUCTION AT DIKE I CAN BE FOUND ON DRAWING 147/2821.
REVISION
DATE
DESCRIPTION
BY
DEPARTMENT OF THE ARMY
LITTLE ROCK DISTRICT, CORPS OF ENGINEERS
LITTLE ROCK, ARKANSAS
DESIGNED BY:
DRAWN BY:
W
CHECKED BY:
WHITE RIVER
ARKANSAS
MAJOR REHABILITATION PROJECT
CONCRETE CUTOFF WALL
LOCATION OF BORINGS
BEAVER DAM - DIKE I
SUBMITTED BY:
ENGINEER
SCALE 1" = 50'
OCTOBER 1991
DRAWING NUMBER
10112-147/3222

PLATE A-11


STATIONS
77+00
76+00
75+00
74+00
73+00
72+00
71+00
70+00
ELEVATION (NGVD)
1180
1160
1140
1120
1100
1080
1060
1040
1020
1000
980
960
940
920
900
880
860
840
820
800
780
760
740
720
700
680
TOP OF WALL EL. 1130
FAULT ZONE
BOTTOM OF WALL
BOTTOM OF GROUT CURTAIN
FAULT ZONE
NOTES:
SYMBOL BESIDE LINE DENOTE THE GROUT TAKE PER LINEAR FOOT OF GROUT HOLE (c.f./ft.) DURING THE INITIAL (1959-60) AND THE REMEDIAL (1968-71) GROUTING PROGRAMS.
GROUT TAKES:
5-10 c.f./ft.
10-15 c.f./ft.
>-15 c.f./ft.
LEGEND
ST. JOE
EXTENT OF CUTOFF WALL



THIS DRAWING HAS BEEN REDUCED TO ONE-HALF THE ORIGINAL SCALE



PLAN VIEW OF EXPLORATORY BORINGS (THIS SHEET ONLY)
DIKE I
N.T.S.

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
LITTLE ROCK DISTRICT, CORPS OF ENGINEERS
LITTLE ROCK, ARKANSAS

DESIGNED BY: DH
DRAWN BY: JTD
CHECKED BY: CD

WHITE RIVER — ARKANSAS

MAJOR REHABILITATION PROJECT
CONCRETE CUTOFF WALL

PROPOSED CUTOFF WALL

BEAVER DAM – DIKE I

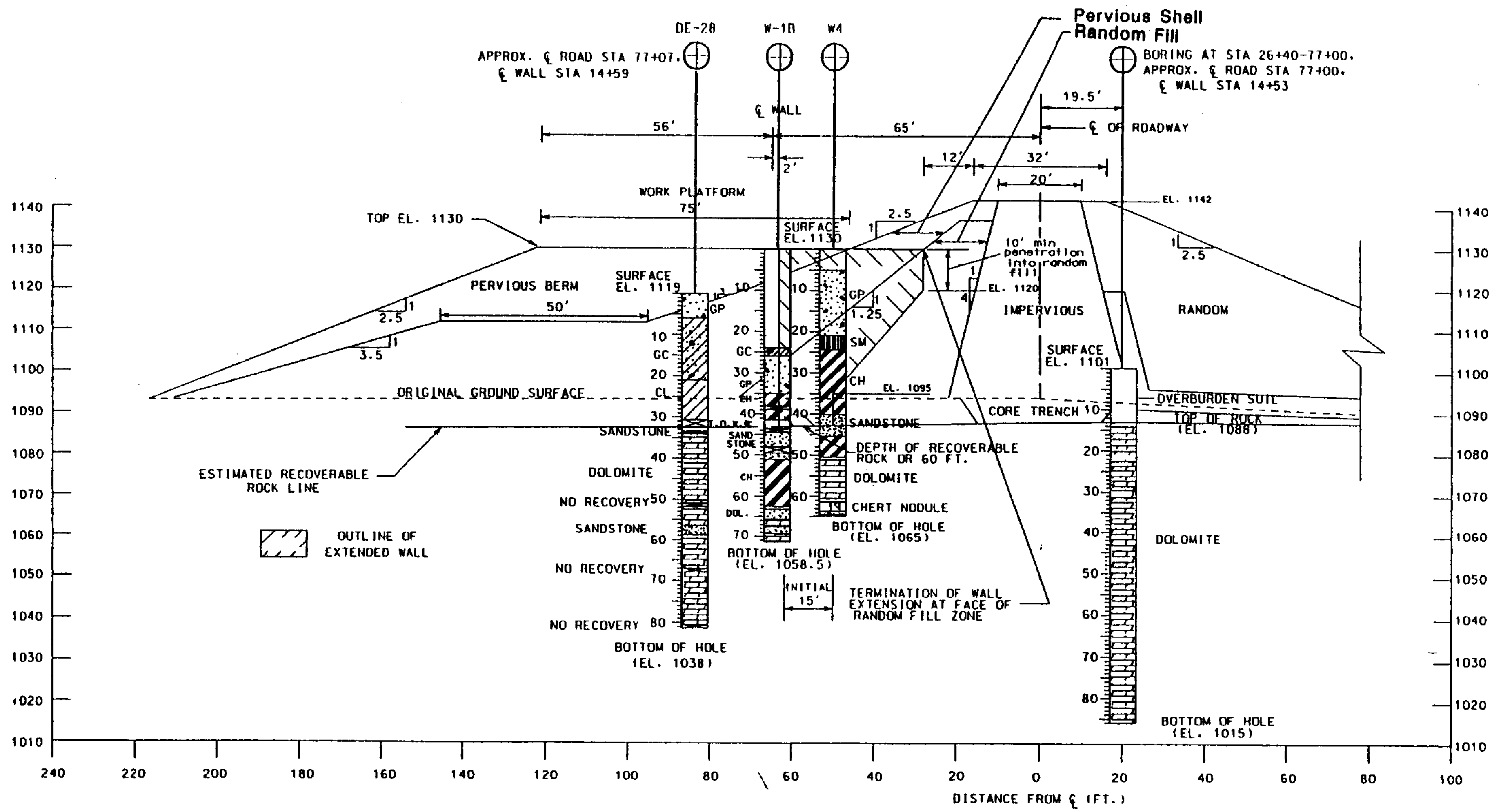
SUBMITTED BY: Carroll D. Winter
ENGINEER

SCALE: AS SHOWN

OCTOBER 1991

SERIAL: 92-R-0001

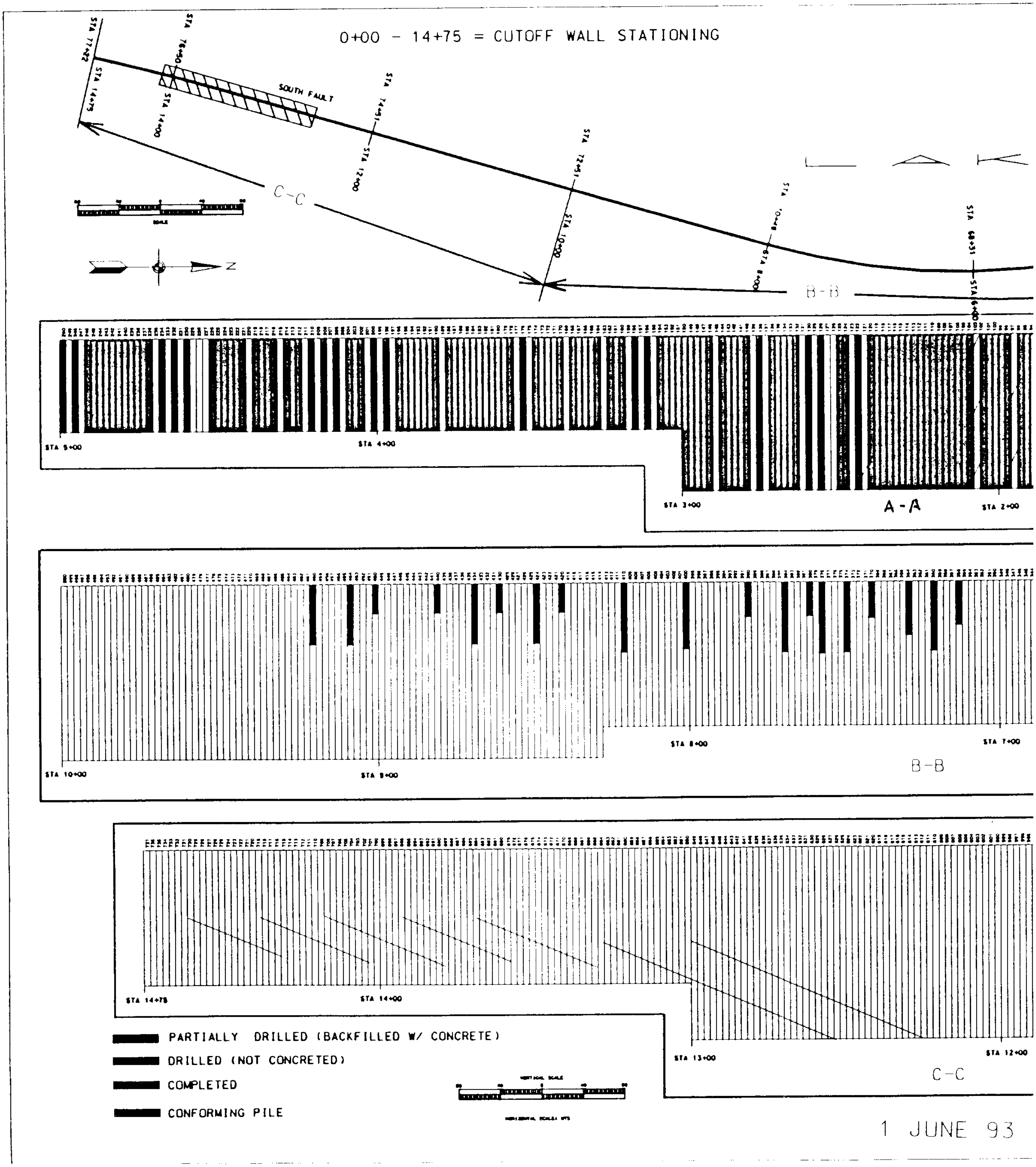
18112-147/ 3223



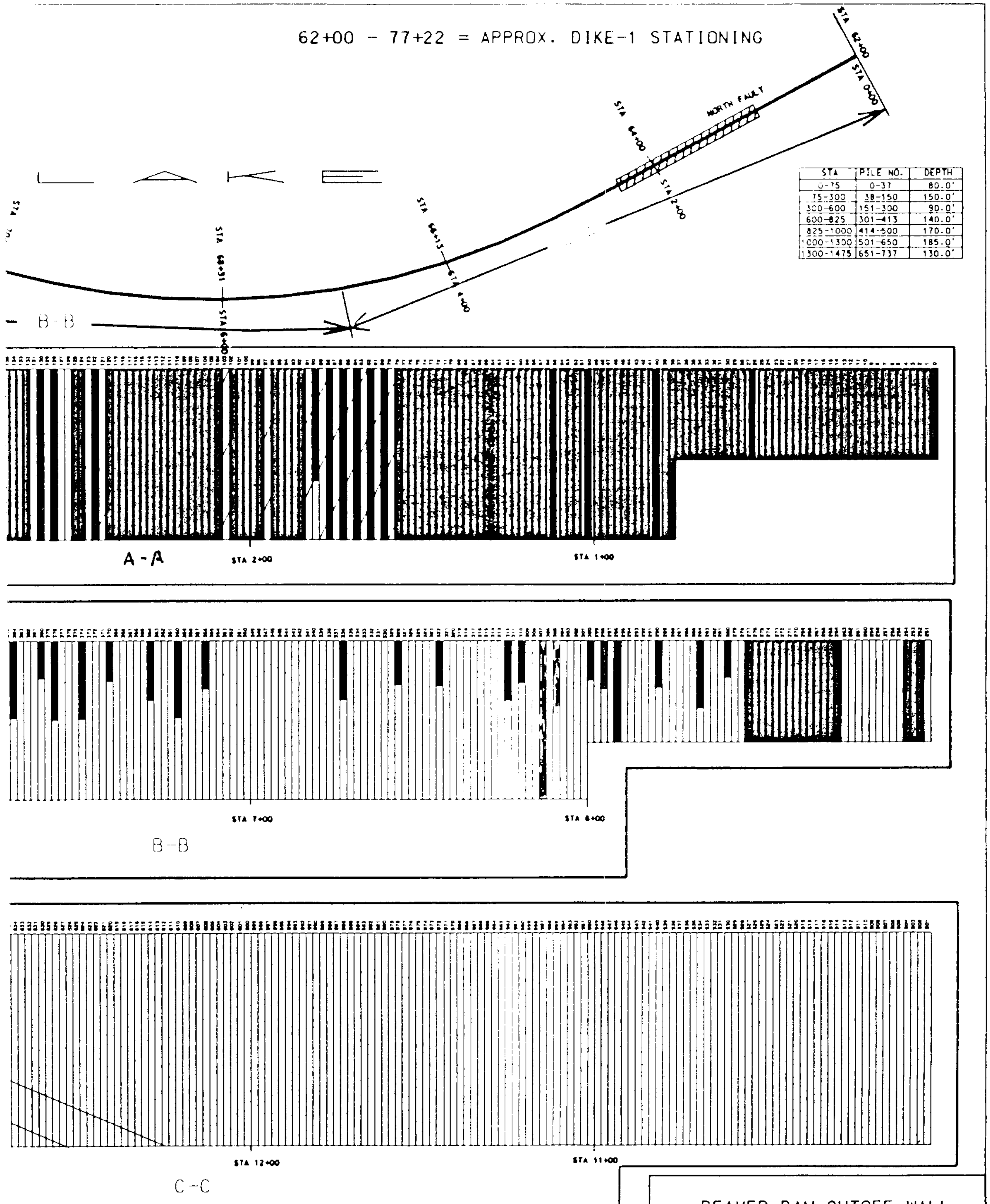
MAIN EMBANKMENT SECTION, (M.E.S.) - GEOLOGICAL PROFILE OF WALL EXTENSION INTO EMBANKMENT, OVERBURDEN, WEATHERED ROCK

(℄ WALL STA 14+75/APPROX. ℄ ROAD STA 77+23)

WORK SAFELY


0+00 - 14+75 = CUTOFF WALL STATIONING
SOUTH FAULT
C-C
B-B
A-A
STA 5+00
STA 4+00
STA 3+00
STA 2+00
STA 10+00
STA 9+00
STA 8+00
STA 7+00
STA 14+75
STA 14+00
STA 13+00
STA 12+00
PARTIALLY DRILLED (BACKFILLED W/ CONCRETE)
DRILLED (NOT CONCRETED)
COMPLETED
CONFORMING PILE
1 JUNE 93

62+00 - 77+22 = APPROX. DIKE-1 STATIONING



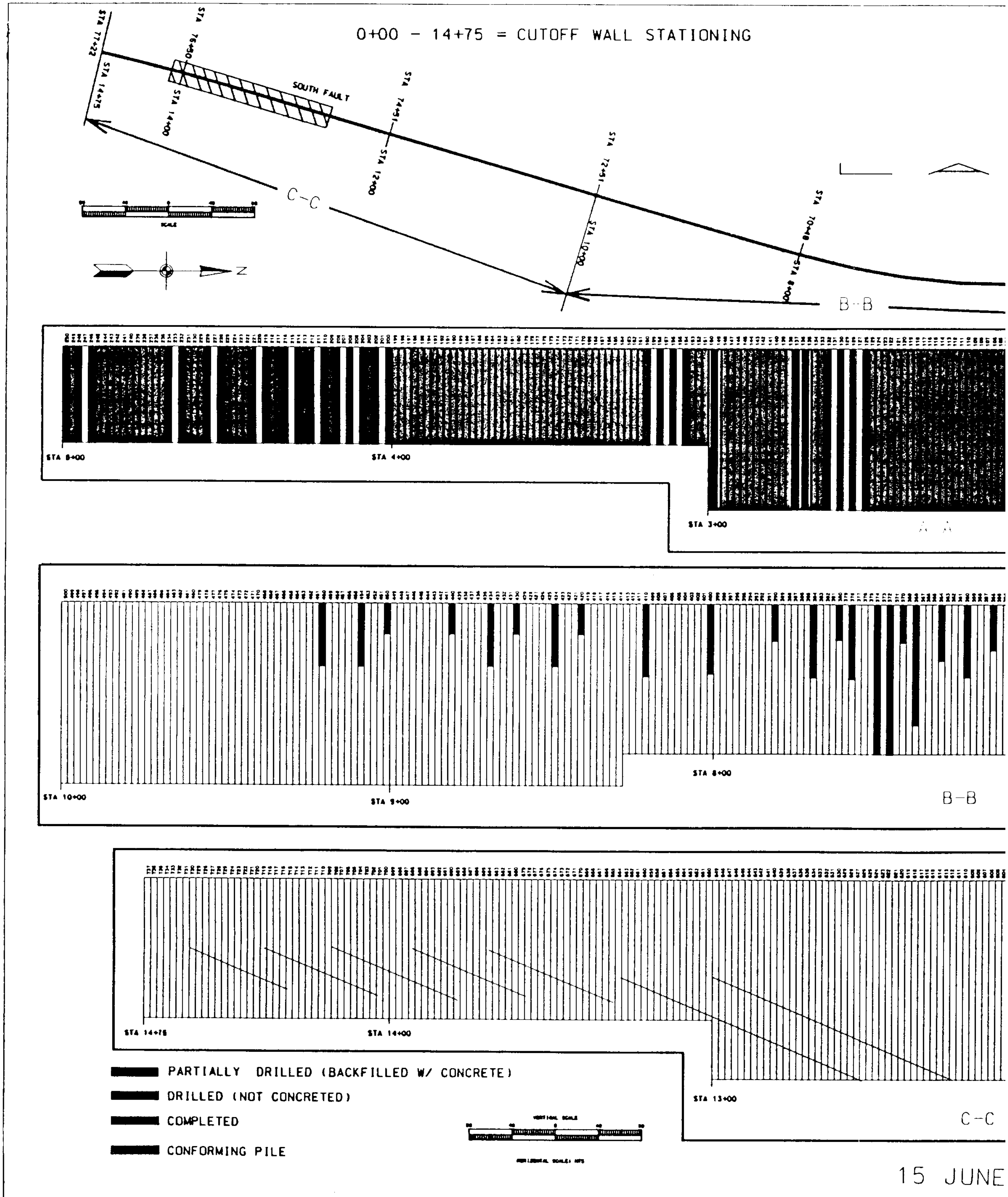
STA	PILE NO.	DEPTH
0-75	0-37	80.0'
75-300	38-150	150.0'
300-600	151-300	90.0'
600-825	301-413	140.0'
825-1000	414-500	170.0'
1000-1300	501-650	185.0'
1300-1475	651-737	130.0'

1 JUNE 93

BEAVER DAM CUTOFF WALL

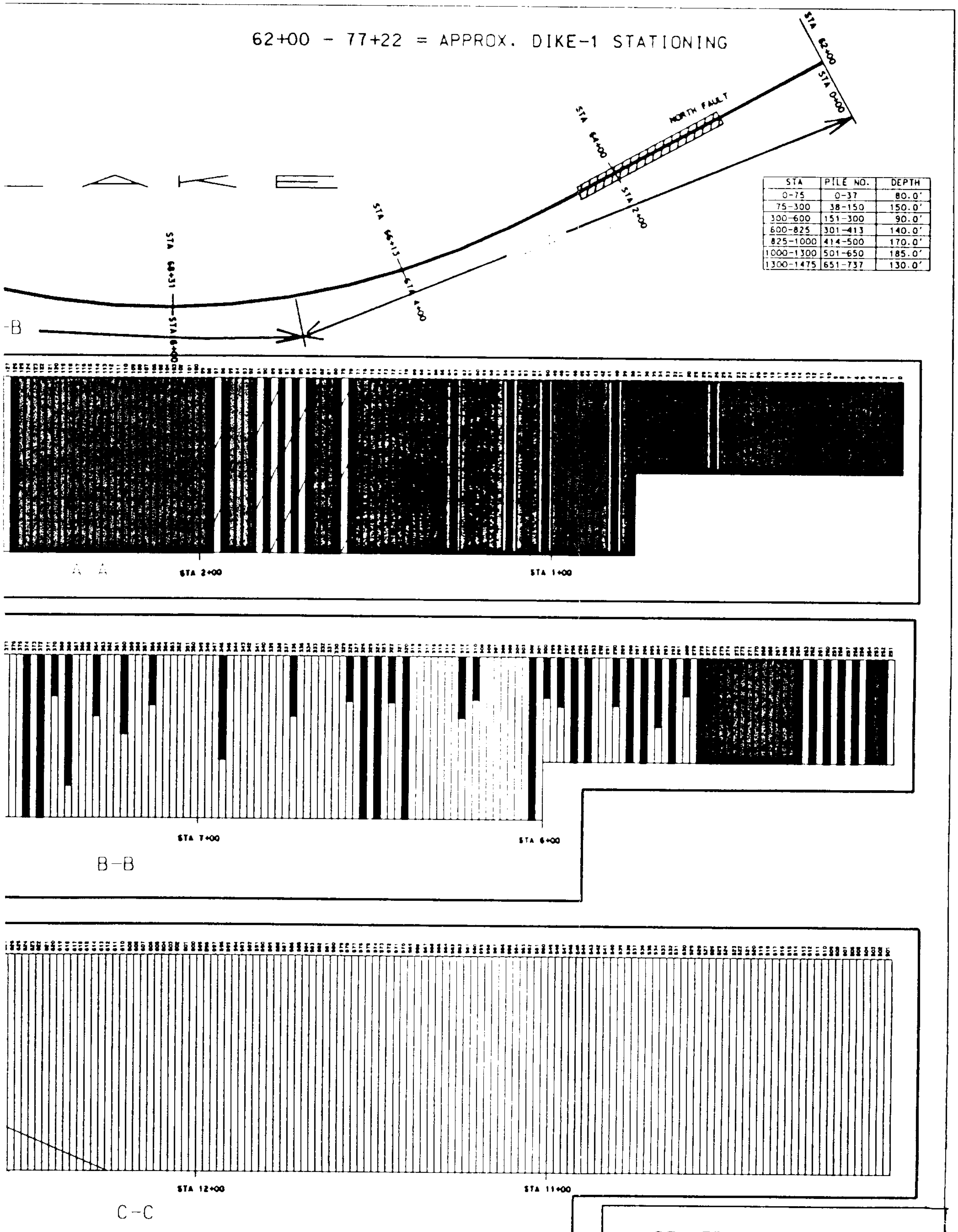
PROFILE

PLATE A-14



0+00 - 14+75 = CUTOFF WALL STATIONING
SOUTH FAULT
STA 14+75
STA 14+00
STA 12+00
STA 10+00
STA 8+00
C-C
B-B
STA 4+00
STA 3+00
STA 9+00
STA 13+00
PARTIALLY DRILLED (BACKFILLED W/ CONCRETE)
DRILLED (NOT CONCRETED)
COMPLETED
CONFORMING PILE
15 JUNE

62+00 - 77+22 = APPROX. DIKE-1 STATIONING



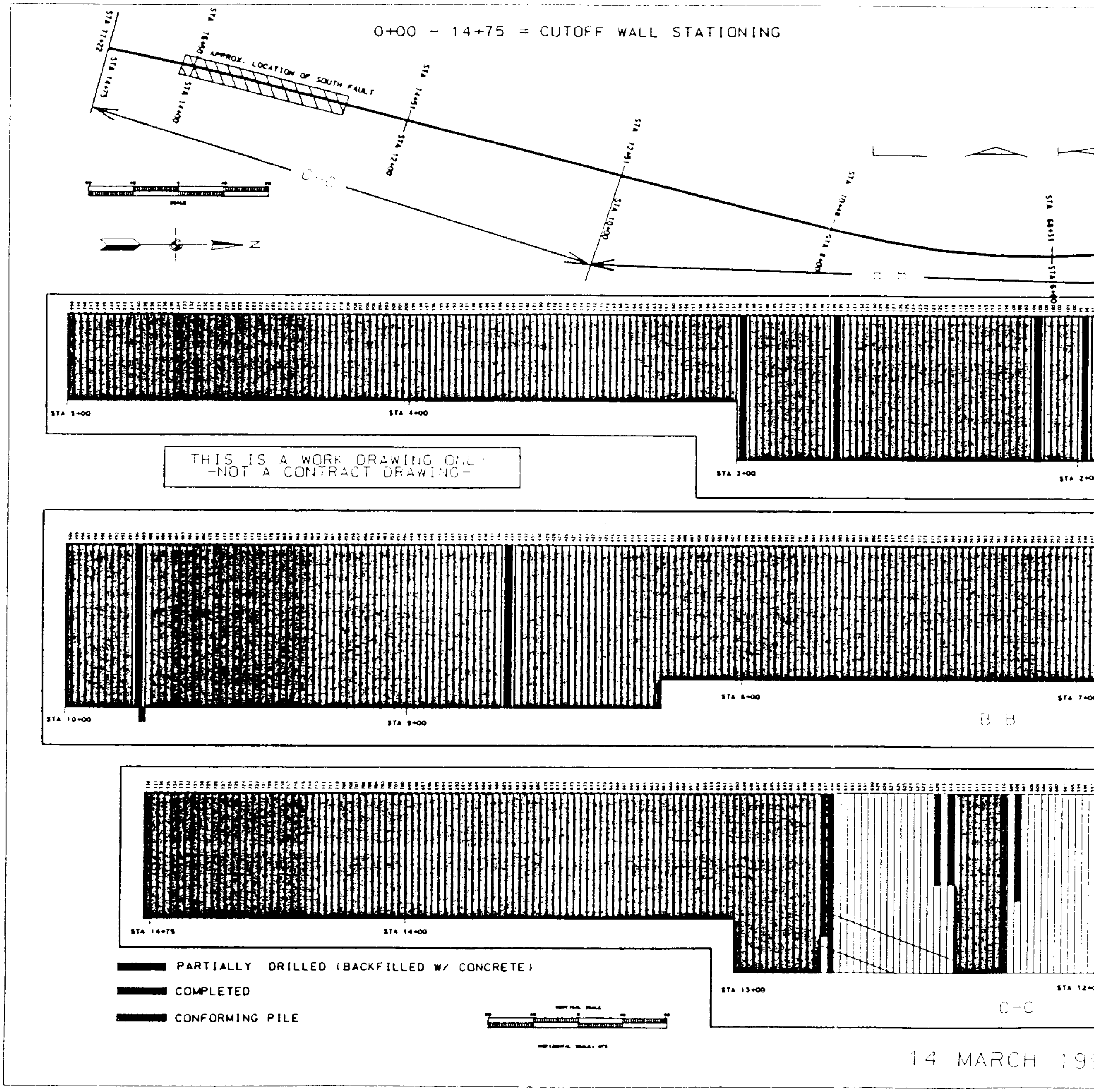
STA	PILE NO.	DEPTH
0-75	0-37	80.0'
75-300	38-150	150.0'
300-600	151-300	90.0'
600-825	301-413	140.0'
825-1000	414-500	170.0'
1000-1300	501-650	185.0'
1300-1475	651-737	130.0'

15 JUNE 93

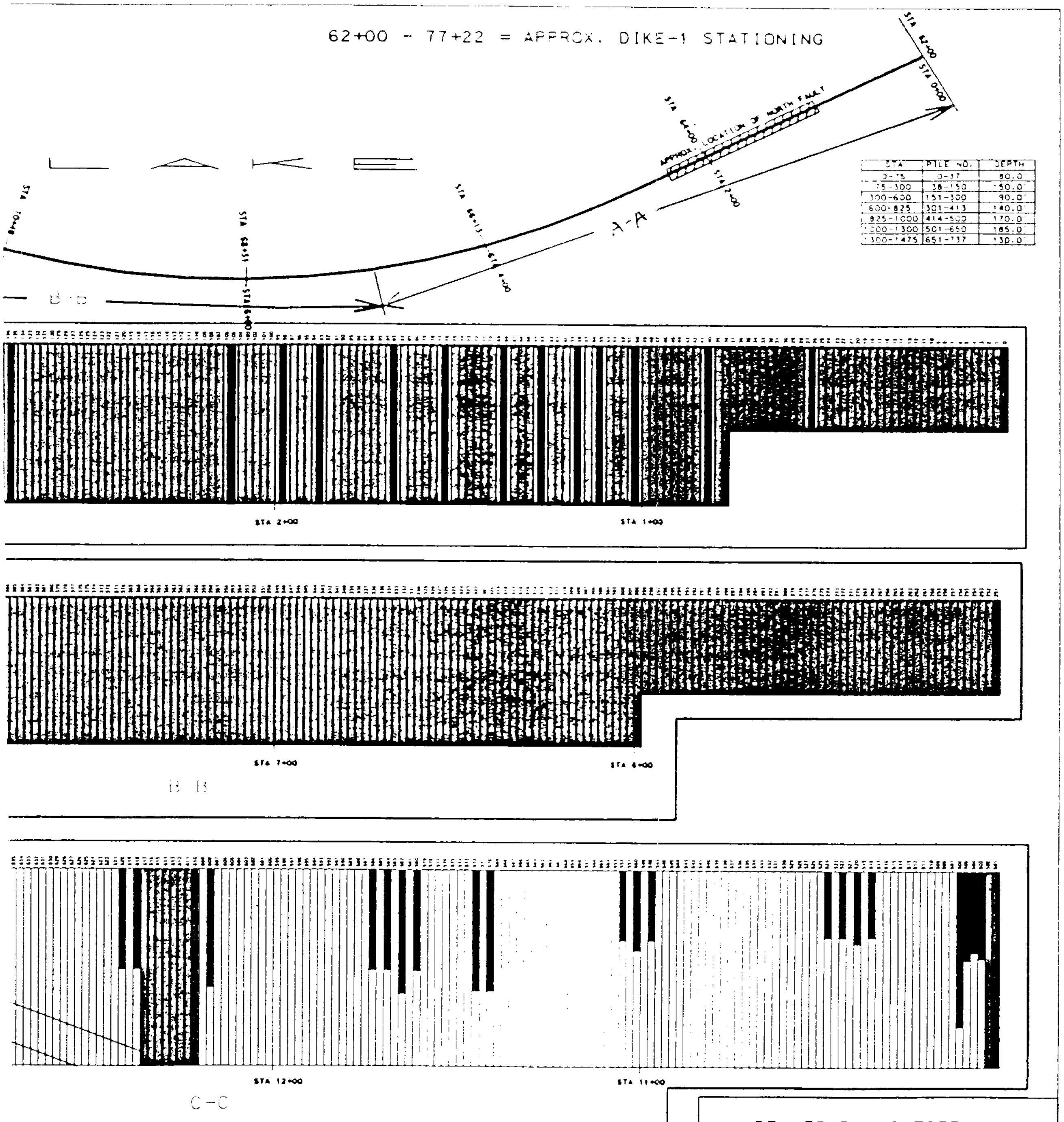
BEAVER DAM CUTOFF WALL

PROFILE

PLATE A-15



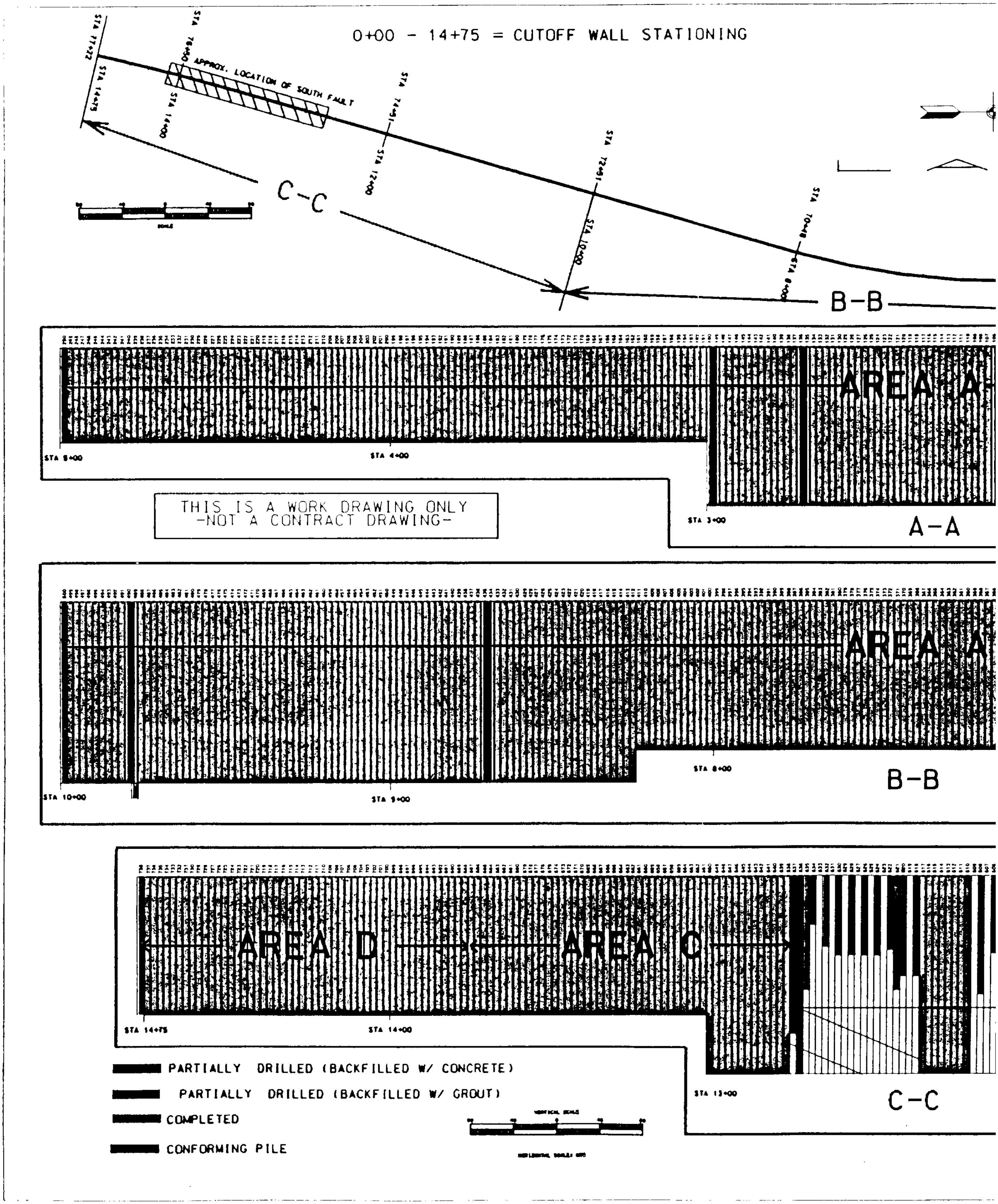
0+00 - 14+75 = CUTOFF WALL STATIONING
APPROX. LOCATION OF SOUTH FAULT
C-C
B-B
THIS IS A WORK DRAWING ONLY
-NOT A CONTRACT DRAWING-
STA 5+00
STA 4+00
STA 3+00
STA 10+00
STA 9+00
STA 8+00
B-B
STA 14+75
STA 14+00
STA 13+00
PARTIALLY DRILLED (BACKFILLED W/ CONCRETE)
COMPLETED
CONFORMING PILE
C-C
14 MARCH 19

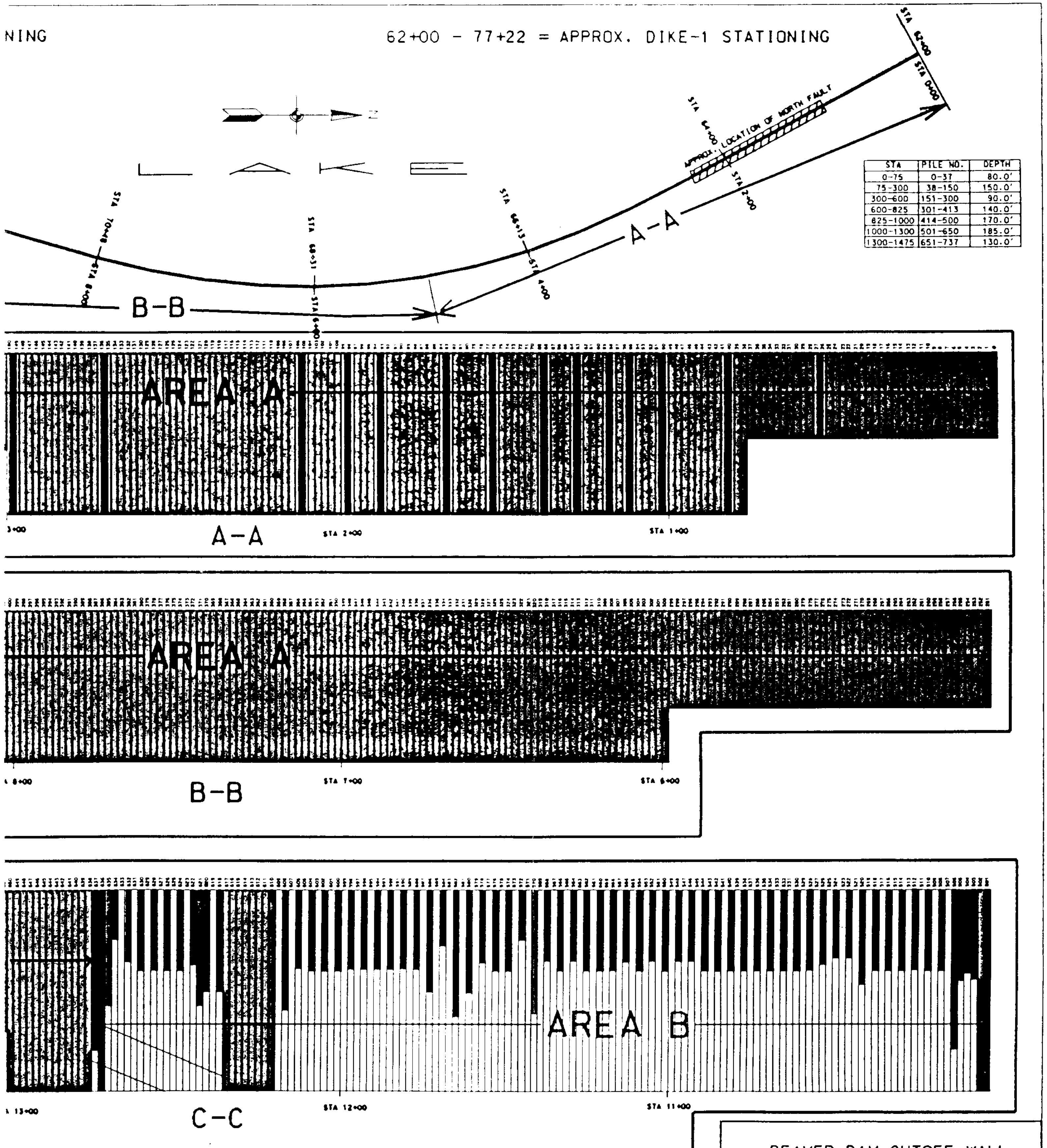


STA	PILE NO.	DEPTH
0-75	0-37	80.0'
75-300	38-150	150.0'
300-600	151-300	90.0'
600-825	301-413	140.0'
825-1000	414-500	170.0'
1000-1300	501-650	185.0'
1300-1475	651-737	130.0'

14 MARCH 1994

BEAVER DAM CUTOFF WALL
PROFILE


0+00 - 14+75 = CUTOFF WALL STATIONING
APPROX. LOCATION OF SOUTH FAULT
C-C
B-B
AREA A
STA 5+00
STA 4+00
STA 3+00
A-A
THIS IS A WORK DRAWING ONLY
-NOT A CONTRACT DRAWING-
AREA A
STA 10+00
STA 9+00
STA 8+00
B-B
AREA D
AREA C
STA 14+75
STA 14+00
STA 13+00
C-C
PARTIALLY DRILLED (BACKFILLED W/ CONCRETE)
PARTIALLY DRILLED (BACKFILLED W/ GROUT)
COMPLETED
CONFORMING PILE



STA	PILE NO.	DEPTH
0-75	0-37	80.0'
75-300	38-150	150.0'
300-600	151-300	90.0'
600-825	301-413	140.0'
825-1000	414-500	170.0'
1000-1300	501-650	185.0'
1300-1475	651-737	130.0'

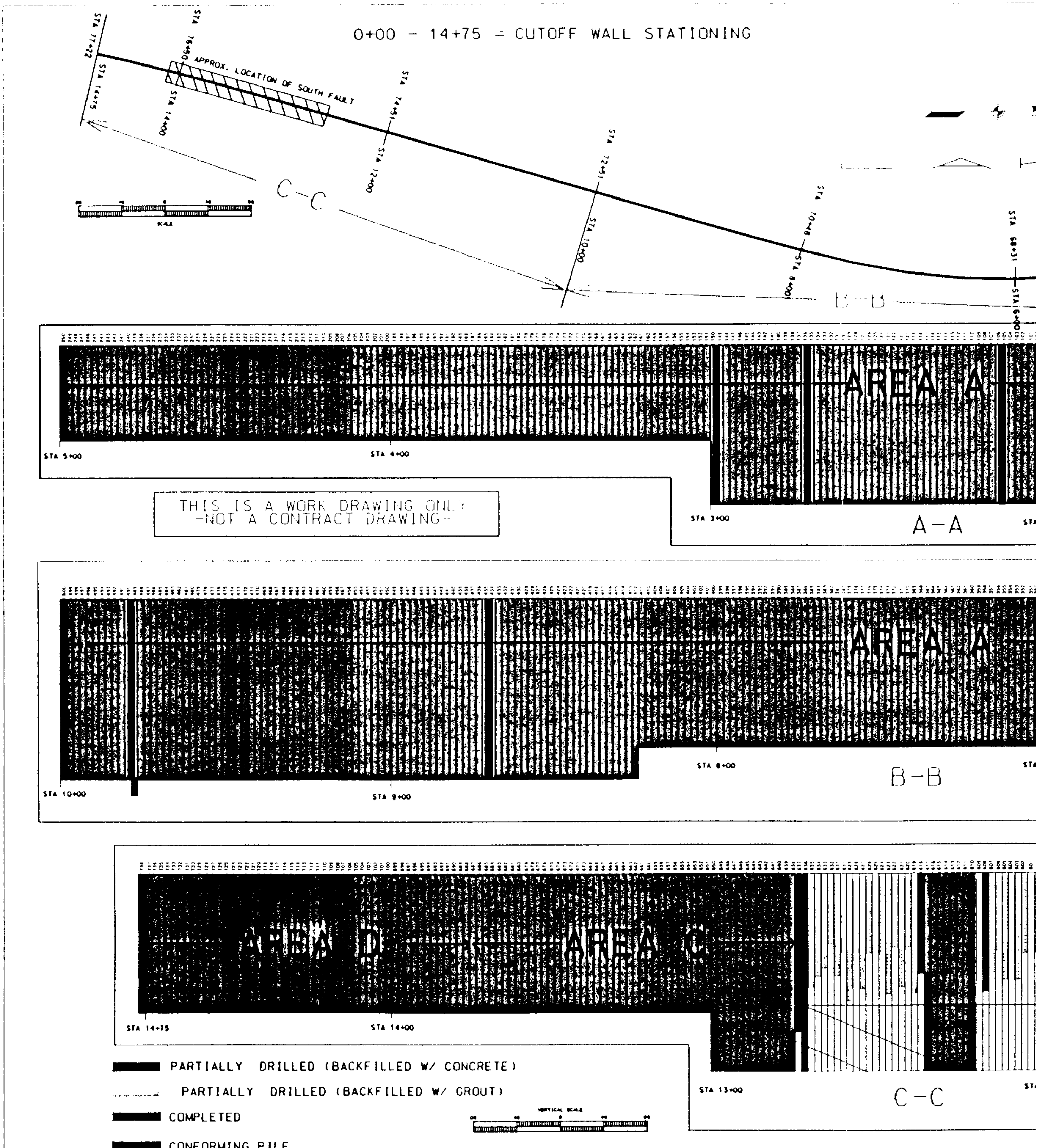
4 April 94

BEAVER DAM CUTOFF WALL

PROFILE

PLATE A-17

0+00 - 14+75 = CUTOFF WALL STATIONING



THIS IS A WORK DRAWING ONLY
-NOT A CONTRACT DRAWING-

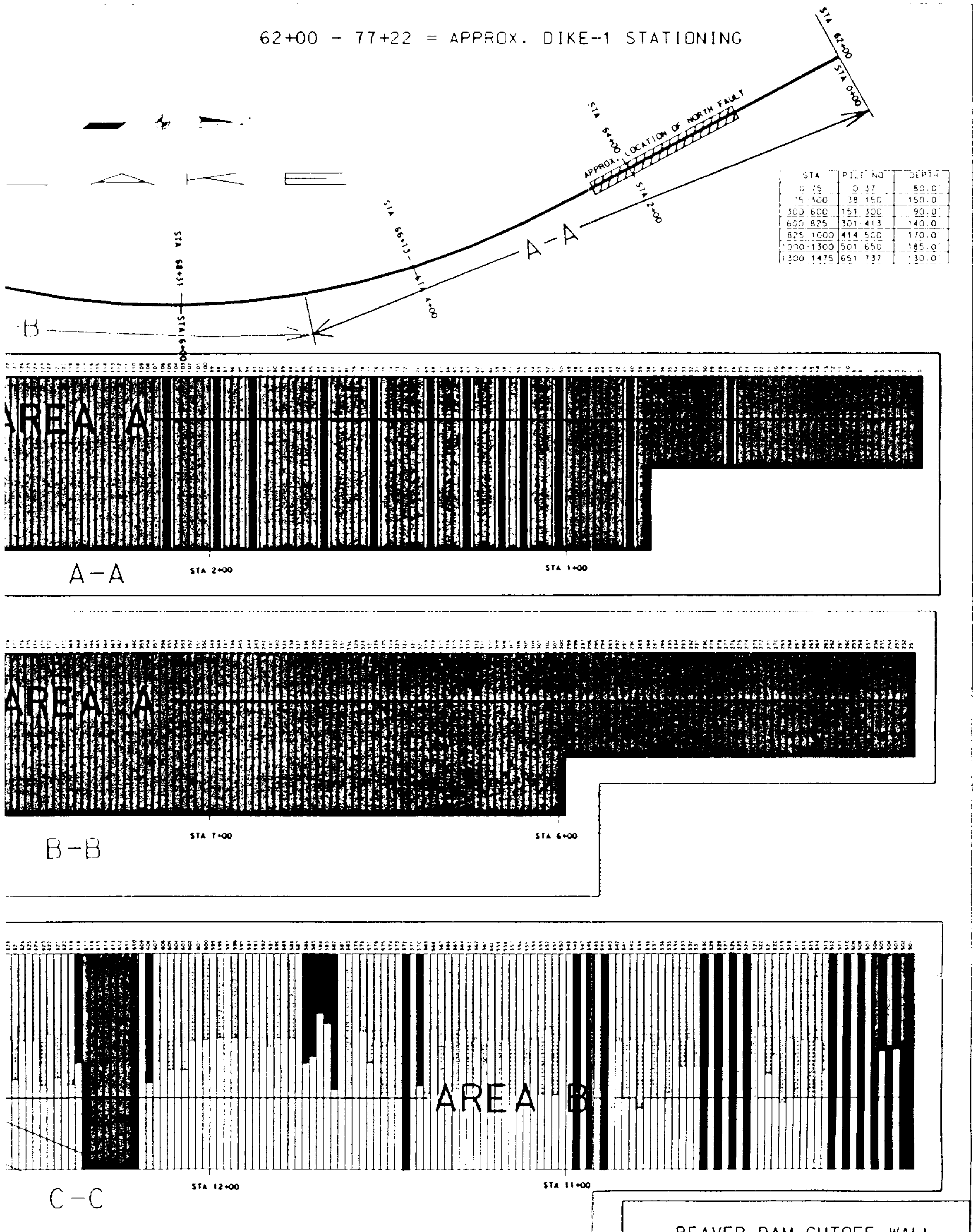
PARTIALLY DRILLED (BACKFILLED W/ CONCRETE)

PARTIALLY DRILLED (BACKFILLED W/ GROUT)

COMPLETED

CONFORMING PILE

11 MAY 1994



STA	PILE NO.	DEPTH
0 75	0 37	80.0
75 300	38 150	150.0
300 600	151 300	90.0
600 825	301 413	140.0
825 1000	414 500	170.0
1000 1300	501 650	185.0
1300 1475	651 737	130.0

1 MAY 1994

BEAVER DAM CUTOFF WALL

PROFILE

PLATE A-18

QUALITY CORING AND TESTING SUMMARY

Pile No.	Chainage	CORING			WATER PRESSURE TESTING				BACKFILLING		
		Date	NX-core (ft.)	6"-core (ft.)	Depth interval (ft.)	Pressure (psi)	Time (min.)	Water take (gal.)	Theor. vol. (cu.ft.)	Grout vol. (cu.ft.)	Wastage (cu.ft.)
40	0+80	01/08/93	156.5		2.0 -156.5	40	5	0.0	7.58	9.04	1.46
82	1+64	01/23/93	156		2.0 - 153.0	40	5	0.0	7.5	9.04	1.54
16-17	0+33	01/25/93		30	2.0 - 30.0	15	5	0.0	9.84	10.17	0.33
188	3+76	02/03/93	89.8		2.0 - 87.0	40	5	0.0	4.21	5.65	1.44
136	2+72	02/08/93	152		Pile #136 redrilled by d.h.h. 34" due to no overlap at full depth w/ pile #135						
4	0+08	02/10/93	82		2.0 - 87.0	40	5	0.0	3.97	4.52	0.55
174	3+48	02/12/93	92		2.0 - 92.0	40	5	0.0	4.45	5.56	1.11
42	0+84	04/22/93	101		2.0 - 101.0	40	5	0.0	4.89	5.65	0.76
46-47	0+93	04/28/93		29.7	2.0 - 29.7	15	5	0.0	9.74	10.17	0.43
62	1+24	04/30/93	160		2.0 - 160.0	40	5	0.0	7.02	7.91	0.89
73-74	1+47	05/01/93		24	2.0 - 24.0	15	10	13.4	7.87	7.91	0.04
258	5+16	07/08/93	93.3		0.0 - 85.0	40	5	0.0	4.51	6.78	2.27
206	4+12	07/10/93	97		2.0 - 85.0	40	5	0.0	4.21	5.65	1.44
83	1+66	07/13/93	0-59.8**		NX-coring began in pile #83 and migrated into pile #84 at 59.80 feet depth						
84	1+68	07/13/93	**59.8-156.6		2.0 - 156.6	40	5	2.6	7.58	9.04	1.46
250	5+00	07/15/93	97		0.0 - 85.0	40	5	0.5	4.69	6.78	2.09
164	3+28	08/14/93	82		2.0 - 82.0	40	5	0.0	3.97	5.65	1.68
302	6+05	08/18/93	144.1		0.0 - 138.0	40	5	0.2	6.97	11.3	4.33
226-227	4+53	08/19/93		30.3	2.0 - 30.3	40	5	0.0	9.94	11.3	1.36
140-141	2+81	09/22/93		30.3	2.0 - 30.3	40	5	0.0	9.94	11.3	1.36
200-201	4+01	09/23/93		31	2.0 - 31.0	40	5	0.0	10.17	11.3	1.13
264-265	5+29	09/24/93		30.9	3.0 - 30.9	40	5	0.0	10.14	11.3	1.16
298-299	5+97	09/27/93		30.1	2.0 - 30.1	40	5	0.0	9.87	11.3	1.43
336-337	6+73	09/28/93		29.8	4.0 - 29.8	40	5	0.0	9.77	11.3	1.53

RODIO-NICHOLSON JOINT VENTURE
BEAVER DAM PROJECT

QUALITY CORING AND TESTING SUMMARY

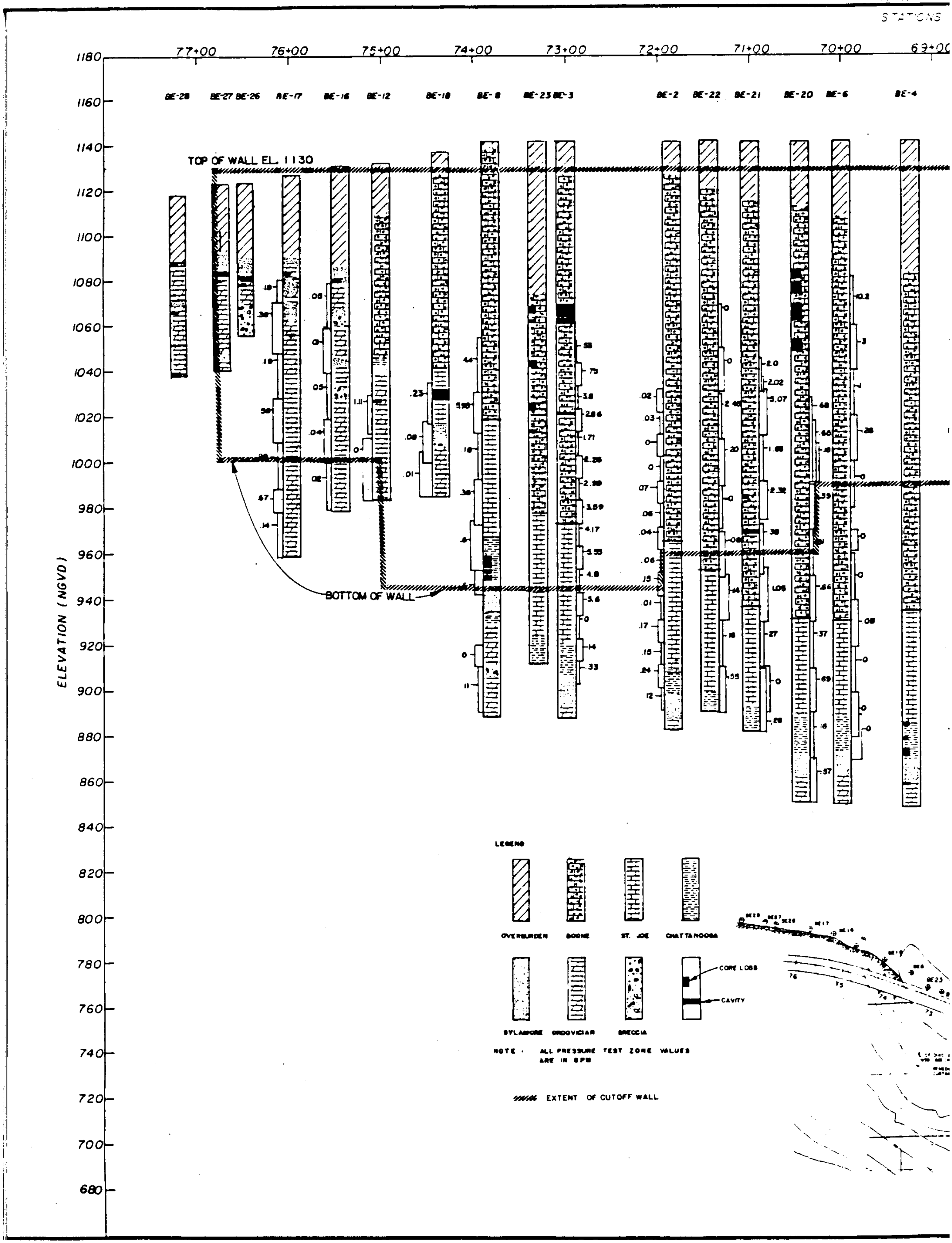
Pile No.	Chainage	CORING			WATER PRESSURE TESTING				BACKFILLING		
		Date	NX-core (ft.)	6"-core (ft.)	Depth interval (ft.)	Pressure (psi)	Time (min.)	Water take (gal.)	Theor. vol. (cu.ft.)	Grout vol. (cu.ft.)	Wastage (cu.ft.)
400-401	8+01	10/26/93		30	4.0 - 30.0	15	5	8.9	9.84	11.3	1.46
368-369	7+37	10/28/93		29.9	4.0 - 29.9	40	5	0.0	9.8	10.2	0.4
430-431	8+61	10/29/93		29.6	4.0 - 29.6	15	5	0.5	9.7	10.2	0.5
470-471	9+41	10/30/93		29.6	4.0 - 29.6	15	5	0.0	9.7	11.3	1.6
412	8+24	11/04/93	177.2						8.57	9.04	0.47
736	14+72	05/14/94	140		2.0 - 127.0	40	5	0.0	6.78	9.04	2.26
719-720	14+39	05/16/94		29.8					9.78	11.3	1.52
612	12+24	05/18/94	174.9						8.5	9.04	0.54
617-618	12+33	05/18/94		29.7	2.0 - 29.7	15	5	0	9.74	11.3	1.56
675	13+50	08/17/94	138						6.68	9.04	2.36
550-551	11+01	08/20/94		39.3	2.0 - 39.3	15	5	0.2	12.9	13.6	0.7
525	10+50	08/23/94	189		2.0 - 189.0	40	5	0.4	9.15	10.17	1.02
575-576	11+51	08/29/94		34.1					11.2	11.3	0.1
621-622	12+43	09/01/94		44.6					14.6	15.8	1.2
			2478.40	**562.70**					**295.77**	**341.25**	**45.48**

BEAVER DAM COMPLETION REPORT

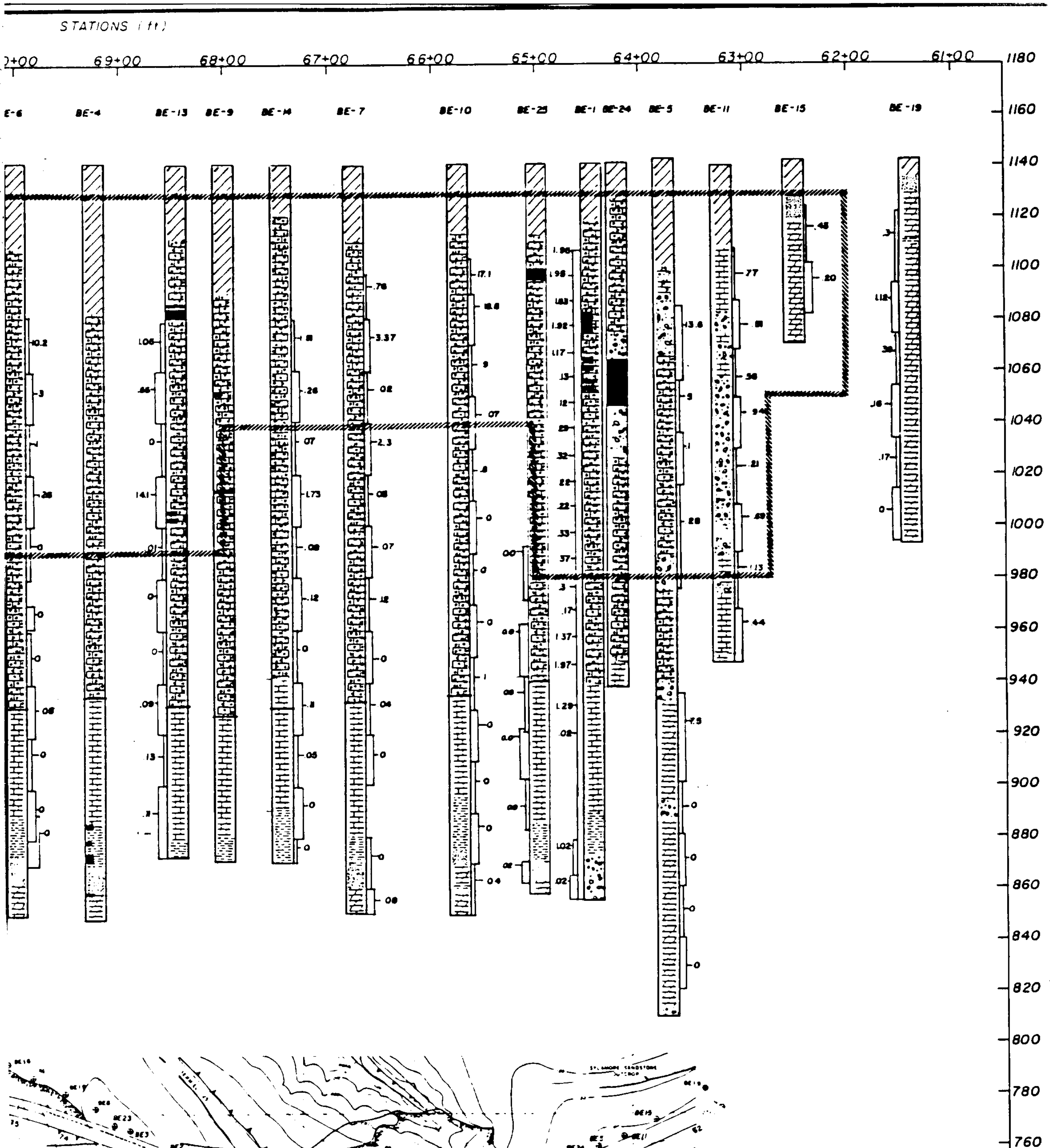
APPENDIX - B

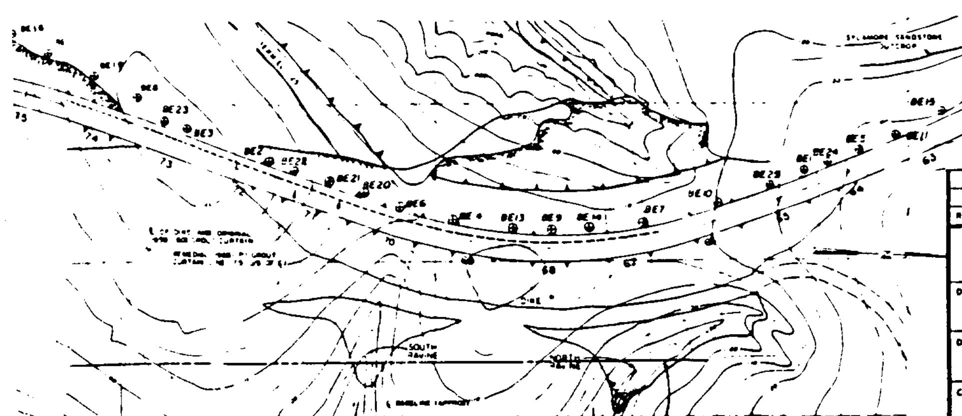
APPENDIX B: GEOLOGICAL & GEOPHYSICAL PLATES AND DRAWINGS

List of Plates



STATIONS
77+00
76+00
75+00
74+00
73+00
72+00
71+00
70+00
BE-28
BE-27
BE-26
BE-17
BE-16
BE-12
BE-18
BE-8
BE-23
BE-3
BE-2
BE-22
BE-21
BE-20
BE-6
BE-4
ELEVATION (NGVD)
1180
1160
1140
1120
1100
1080
1060
1040
1020
1000
980
960
940
920
900
880
860
840
820
800
780
760
740
720
700
680
TOP OF WALL EL. 1130
BOTTOM OF WALL
LEGEND
OVERBURDEN
BOONE
ST. JOE
CHATTANOOGA
SYLAMORE
ORDOVICIAN
BRECCIA
CORE LOSS
CAVITY
NOTE : ALL PRESSURE TEST ZONE VALUES ARE IN GPM
EXTENT OF CUTOFF WALL





PLAN VIEW OF EXPLORATORY BORINGS (THIS SHEET ONLY)

DIKE I

N.T.S.

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
LITTLE ROCK DISTRICT, CORPS OF ENGINEERS
LITTLE ROCK, ARKANSAS

DESIGNED BY: SH	WHITE RIVER — ARKANSAS
DRAWN BY: JTD	MAJOR REHABILITATION PROJECT CONCRETE CUTOFF WALL
CHECKED BY: CD	PRESSURE TESTS
	BEAVER DAM – DIKE I

SUBMITTED BY: Carroll D. Winter, ENGINEER

SCALE: AS SHOWN

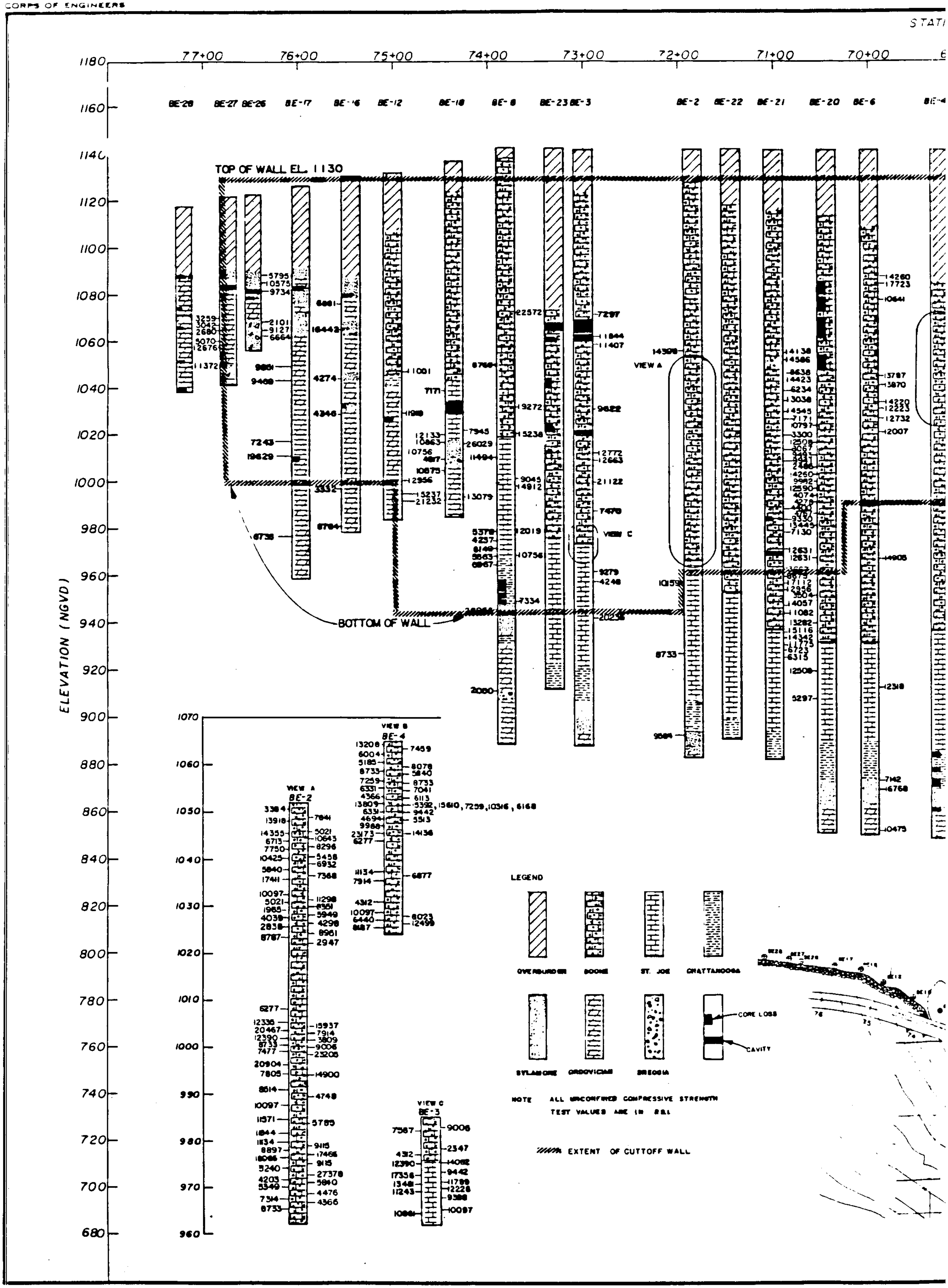
OCTOBER 1991

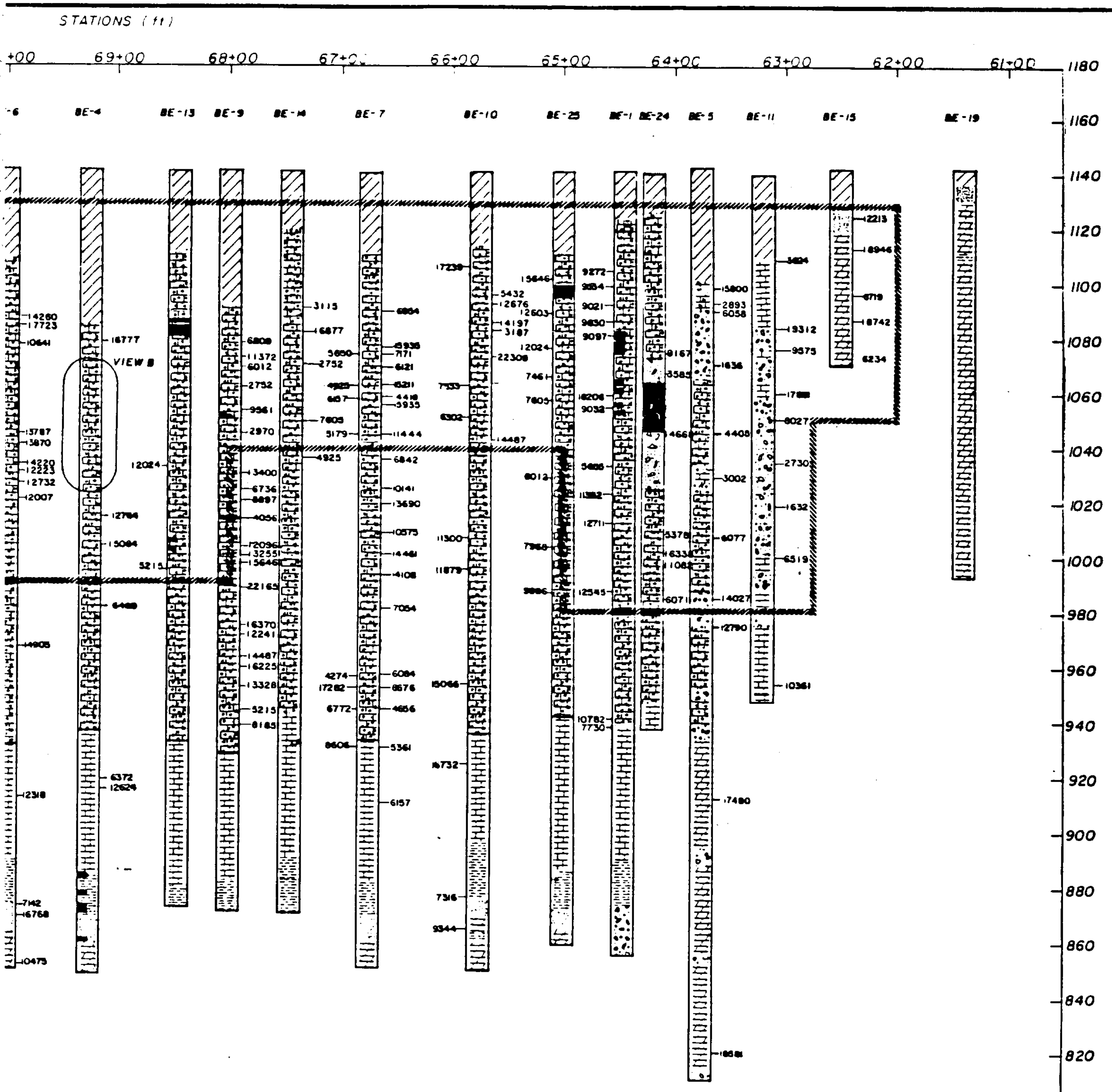
INV. SERIAL NO. 92-R-0007

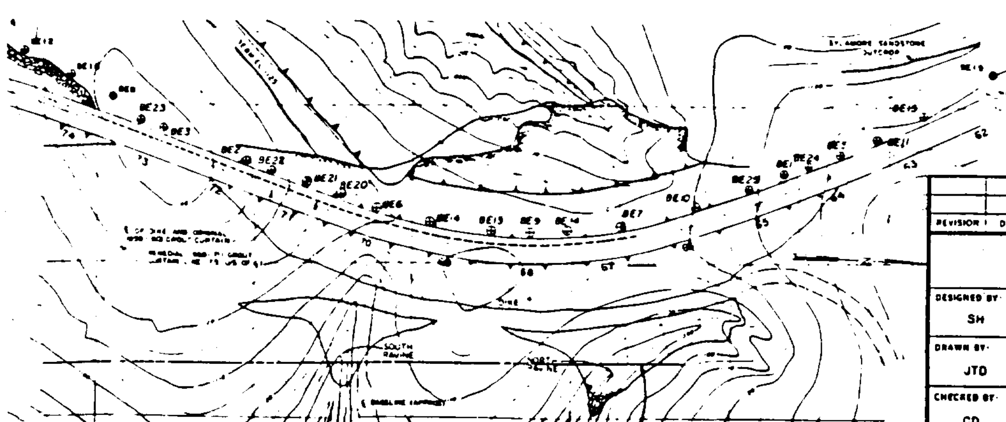
DRAWING NUMBER 10112-147/3225

PLATE B-1

CORPS OF ENGINEERS







PLAN VIEW OF EXPLORATORY BORINGS (THIS SHEET ONLY)

DIKE I

N.T.S.

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
LITTLE ROCK DISTRICT, CORPS OF ENGINEERS
LITTLE ROCK, ARKANSAS

DESIGNED BY: SH	WHITE RIVER — ARKANSAS
DRAWN BY: JTD	MAJOR REHABILITATION PROJECT CONCRETE CUTOFF WALL
CHECKED BY: CD	UNCONFINED COMPRESSIVE STRENGTHS
	BEAVER DAM – DIKE I

SUBMITTED BY: Carroll D. Winter, ENGINEER

SCALE: AS SHOWN

OCTOBER 1991

INV. SERIAL NO. 92-R-0001

DRAWING NUMBER 10112-147/ 3224

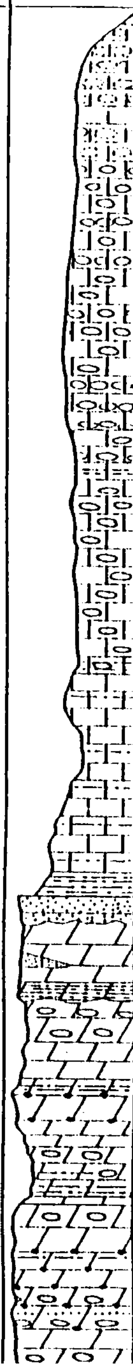
SHEET 7 OF 10

PLATE B-2

DIKE NO. 1

KIRK MOUNTAIN QUARRY SITE

MAIN DAM-EMBANKMENT, DIKE NO. 2, DIKE NO. 3



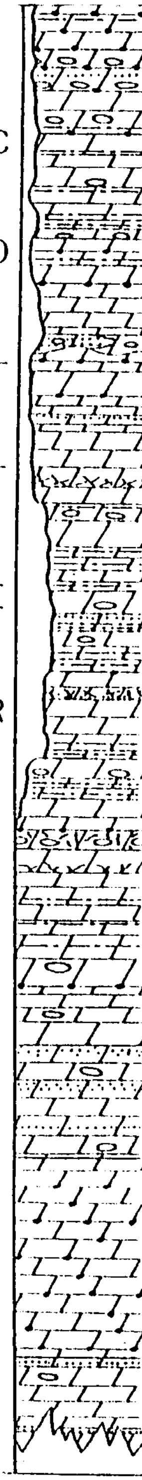
SYSTEM	GROUP	FORMATION	THICKNESS IN FEET	UNIT	DESCRIPTION
MISSISSIPPIAN		BOONE		BOONE CHERT	LIMESTONE, CHERTY, LIGHT TO DARK GRAY, SLIGHTLY SHALY, FINE-GRAINED, HARD, THIN- TO MEDIUM-BEDDED. CHERT OCCURS AS THIN BEDS AND NODULES, SHALE OCCURS AS THIN LAMINATIONS.
MISSISSIPPIAN		BOONE	45 to 50	ST. JOE LIMESTONE	LIMESTONE, GRAY TO GREEN-GRAY WITH OCCASIONAL PINK HORIZONS, CRYSTALLINE, VERY FOSSILIFEROUS, CONTAINS NUMEROUS THIN SHALE SEAMS AND PARTINGS.
DEVONIAN	CHATTANOOGA			CHAT. SHALE	SHALE, BLACK, FIRM, FISSILE.
DEVONIAN				SYLAMORE SS	SANDSTONE, WHITE TO YELLOW, MEDIUM- TO COARSE-GRAINED. VARIES FROM POORLY CEMENTED TO QUARTZITIC. USUALLY MASSIVE, LOCALLY THIN-BEDDED.
O	J E F	POWELL	17±	1 Marker V	LIMESTONE, LIGHT GRAY TO GREEN WHEN FRESH, AND TAN TO BUFF WHEN WEATHERED. IT IS DENSE, MODERATELY HARD, AND MEDIUM- TO MASSIVE-BEDDED. MARKER V, A GREEN ARGILLACEOUS ZONE CONTAINING NUMEROUS SHALE SEAMS AND INCLUSIONS FORMS THE BASE OF THE UNIT.
			21±	2	LIMESTONE, DOLOMITIC, LIGHT GRAY TO GRAY-GREEN WHEN FRESH, AND TAN OR BUFF WHEN WEATHERED. IT IS DENSE AND MODERATELY HARD. THE UPPER HALF CONTAINS SCATTERED CHERT NODULES AND ANGULAR FRAGMENTS, WHICH ARE BANDED BLUE AND GRAY. NEAR THE MIDDLE OF THE UNIT IS A MOTTLED, OFTEN VUGGY ZONE. THE BEDS ARE THICK TO MASSIVE THROUGHOUT MOST OF THE UNIT BUT ARE THIN AND CONTAIN CLOSELY SPACED SHALE SEAMS AT THE BOTTOM.
			26±	3	LIMESTONE, DOLOMITIC, GRAY TO DARK GRAY WHEN FRESH, AND TAN TO BUFF WHEN WEATHERED. THE UPPER HALF OF THE UNIT IS VERY FINELY CRYSTALLINE, HARD, THICK- TO MASSIVE-BEDDED, MOTTLED, AND OCCASIONALLY VUGGY. THE BOTTOM HALF IS DENSE, MEDIUM- TO THICK-BEDDED, AND CONTAINS SCATTERED CHERT NODULES.
			41±	4	LIMESTONE, DOLOMITIC, BLUE-GRAY TO DARK GRAY WHEN FRESH, AND BROWN TO BUFF WHEN WEATHERED. THE PREDOMINANT TYPE OF ROCK IS HARD, MOTTLED, OFTEN VUGGY, AND FINELY CRYSTALLINE, BUT THERE ARE SEVERAL ZONES OF MODERATELY HARD, DENSE ROCK. THE BEDDING IS GENERALLY THICK TO MASSIVE. SCATTERED CHERT NODULES AND BANDS OCCUR THROUGHOUT. THE BASAL PORTION IS COMPOSED OF A DENSE, MODERATELY HARD ROCK WHICH BECOMES VERY SHALY AT ITS BASE. MARKER IV IS THE TOP OF THE SHALY HORIZON.

ORDOVICIAN

JEFFERSON CITY

COTTER

MAIN DAM - CONCRETE



Thickness	Unit	Description
41±	4	... MOTTLED, OFTEN VUGGY, AND FINELY CRYSTALLINE, BUT THERE ARE SEVERAL ZONES OF MODERATELY HARD, DENSE ROCK. THE BEDDING IS GENERALLY THICK TO MASSIVE. SCATTERED CHERT NODULES AND BANDS OCCUR THROUGHOUT. THE BASAL PORTION IS COMPOSED OF A DENSE, MODERATELY HARD ROCK WHICH BECOMES VERY SHALY AT ITS BASE. MARKER IV IS THE TOP OF THE SHALY HORIZON.
	Marker IV	
14±	5	LIMESTONE, DOLOMITIC, DARK GRAY WHEN FRESH, AND TAN OR BUFF WHEN WEATHERED. IT IS VERY FINELY CRYSTALLINE, HARD, AND MEDIUM- TO THICK-BEDDED. THE TOP 6 FEET CONTAINS SCATTERED VUGS.
8±	6	LIMESTONE, DOLOMITIC, GRAY, GRAY-TAN OR BUFF WHEN WEATHERED, DENSE, MODERATELY HARD. TOP IS MEDIUM-BEDDED, BECOMES THIN-BEDDED TOWARD THE BOTTOM.
12±	7	LIMESTONE, DOLOMITIC. TOP AND BOTTOM ZONES GRAY TO BLUE-GRAY, VERY FINELY CRYSTALLINE, VUGGY, MEDIUM TO THICK BEDS. MIDDLE ZONE IS GRAY, DENSE, MODERATELY HARD, CONTAINING CLOSELY SPACED SHALE SEAMS AND PARTINGS.
16±	8	LIMESTONE, DOLOMITIC, LIGHT GRAY TO TAN, DENSE, MODERATELY HARD TO HARD, MEDIUM- TO THICK-BEDDED. IT CONTAINS SANDSTONE AND SANDY ZONES AND OCCASIONALLY A SANDSTONE LIMESTONE BRECCIA NEAR ITS BASE.
30±	9	LIMESTONE, DOLOMITIC. THE UPPER PORTION CONSISTS OF A GRAY TO BLUE-GRAY, FINELY CRYSTALLINE, MEDIUM- TO MASSIVE-BEDDED ROCK WHICH CONTAINS SCATTERED VUGS AND CHERT NODULES. THE LOWER PORTION IS GRAY, MODERATELY HARD TO HARD, AND IS MEDIUM- TO THICK-BEDDED. MARKER III IS THE TOP OF A TAN, DENSE, MODERATELY HARD ZONE AT THE BASE OF THE UNIT. IMMEDIATELY ABOVE MARKER III IS A THIN, WIDESPREAD ZONE OF FRACTURED, OFTEN CAVERNOUS ROCK.
	Marker III	
34±	10	LIMESTONE, DOLOMITIC, GRAY TO BLUE-GRAY WHEN FRESH, AND BROWN TO BUFF WHEN WEATHERED. IT IS PREDOMINANTLY DENSE AND MODERATELY HARD BUT CONTAINS ZONES OF VERY FINELY CRYSTALLINE, HARD ROCK. SCATTERED THROUGHOUT ARE A FEW CHERT NODULES. MOST OF THE UNIT IS THIN- TO MEDIUM-BEDDED. THERE ARE NUMEROUS CLOSELY SPACED SHALE SEAMS AND PARTINGS AT ITS BASE.
20±	11	LIMESTONE, DOLOMITIC, DARK TO LIGHT GRAY. NEAR THE TOP OF THE UNIT THE ROCK IS VERY FINELY CRYSTALLINE, MEDIUM TO THICK-BEDDED, HARD, MOTTLED, AND SOMETIMES VUGGY. IT BECOMES LESS CRYSTALLINE DOWNWARD, AND THE SHALE SEAMS AND PARTINGS BECOME MORE CLOSELY SPACED. AT THE BOTTOM IT IS DENSE AND THIN-BEDDED.
	Marker II	
22±	12	LIMESTONE, DOLOMITIC. THE TOP AND BOTTOM PORTIONS OF THE UNIT ARE GRAY TO BROWN GRAY, VERY FINELY CRYSTALLINE, HARD, MEDIUM- TO THICK-BEDDED, AND OCCASIONALLY VUGGY AND CHERTY. THESE PORTIONS ARE OFTEN SILICIFIED AND FRACTURED. MARKER II IS A THIN-BEDDED HORIZON WHICH SEPARATES THE UPPER AND LOWER PORTIONS OF THE UNIT.
13±	13	LIMESTONE, DOLOMITIC, GRAY TO LIGHT GRAY, VERY FINELY CRYSTALLINE TO DENSE, HARD, MEDIUM TO MASSIVE-BEDDED. IT CONTAINS SCATTERED CHERT NODULES AND VUGS. MARKER I IS A BROWN, VERY HARD, SILICIFIED BED NEAR THE BASE OF THE UNIT. THREE ARGILLACEOUS SEAMS 2 TO 5 INCHES THICK OCCUR WITHIN THE UNIT. THE UPPER TWO SEAMS ARE NOT PERSISTENT AND OFTEN PINCH OUT LATERALLY WITHIN VERY SHORT DISTANCES. THE LOWEST, WHICH OCCURS 3 TO 4 FEET ABOVE THE TOP OF MARKER I, IS PRESENT UNDER ALL OF THE VALLEY BOTTOM. OVER THE GREATER PART OF THE AREA IT IS UNWEATHERED BUT SHOWS SIGNS OF BEING DISTURBED. AT ITS CONTACT WITH THE OVERLYING LIMESTONE THERE IS A VERY THIN, WIDESPREAD OPEN HORIZON. IN THE AREA UPSTREAM FROM THE POWERHOUSE THIS SEAM IS WEATHERED.
	Marker I	
55±	14	LIMESTONE, DOLOMITIC, GRAY TO BLUE GRAY, VERY FINELY CRYSTALLINE TO DENSE, HARD, MEDIUM TO MASSIVE-BEDDED. SCATTERED SHALE SEAMS, STYLOLITES, AND PARTINGS ARE PRESENT THROUGHOUT. THE LOWER PORTION OF THE UNIT CONTAINS SANDY ZONES, BEDS OF SANDSTONE, AND SCATTERED CHERT NODULES.
46±	15	LIMESTONE, DOLOMITIC, GRAY, VERY FINELY CRYSTALLINE, HARD, MEDIUM TO THICK-BEDDED. VARIES FROM SLIGHTLY VUGGY TO VUGGY.
20±	16	LIMESTONE, DOLOMITIC, GRAY, FINELY CRYSTALLINE TO DENSE, HARD. SCATTERED SHALE SEAMS, PARTINGS, SANDY ZONES, AND CHERT NODULES OCCUR THROUGHOUT THE UNIT.

WHITE RIVER WATERSHED WHITE RIVER, ARKANSAS

BEAVER RESERVOIR
GENERALIZED GEOLOGIC COLUMN

U.S. ARMY ENGINEER DISTRICT, LITTLE ROCK
LITTLE ROCK, ARKANSAS, JULY 1959
SCALE AS SHOWN

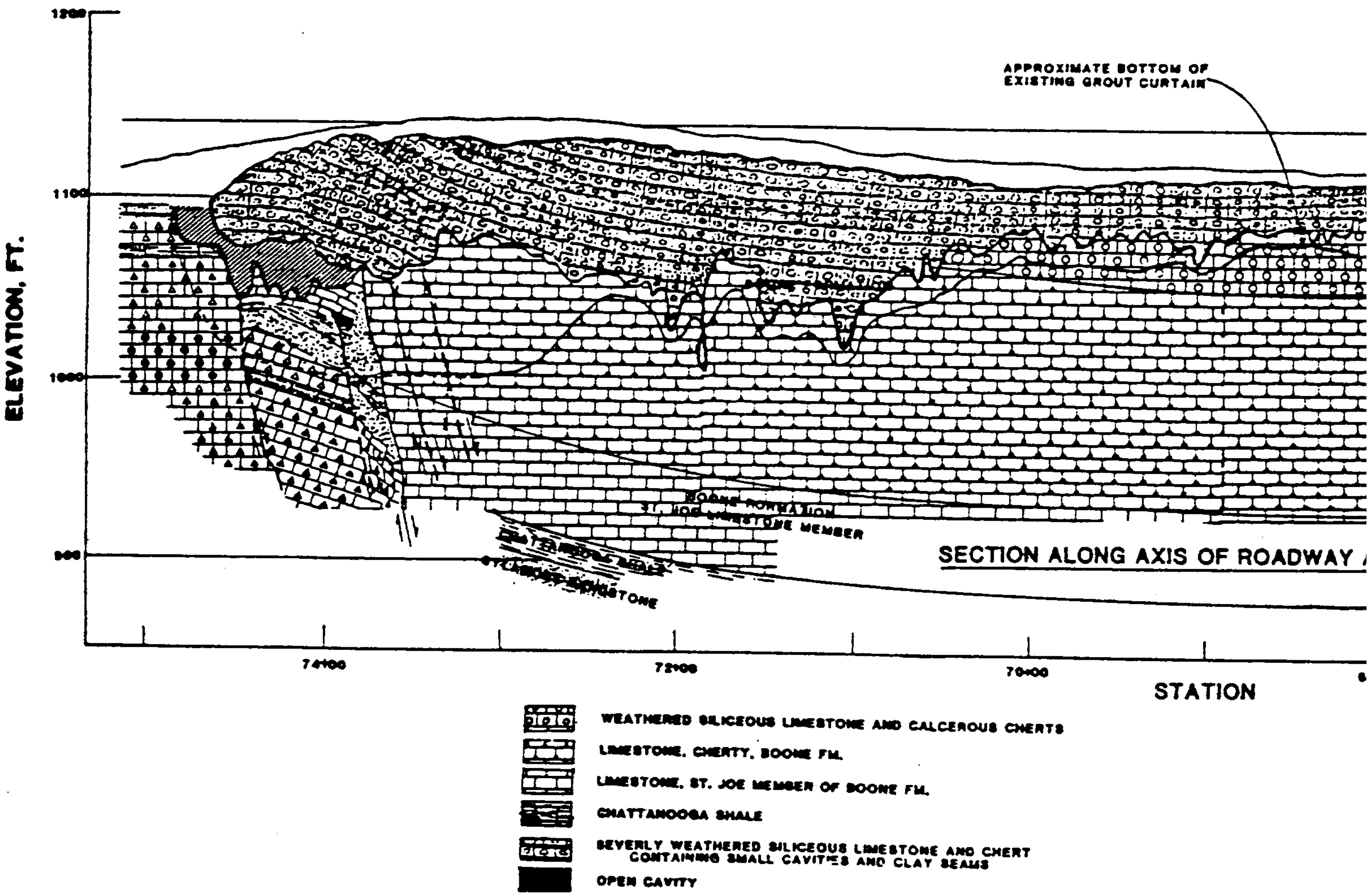
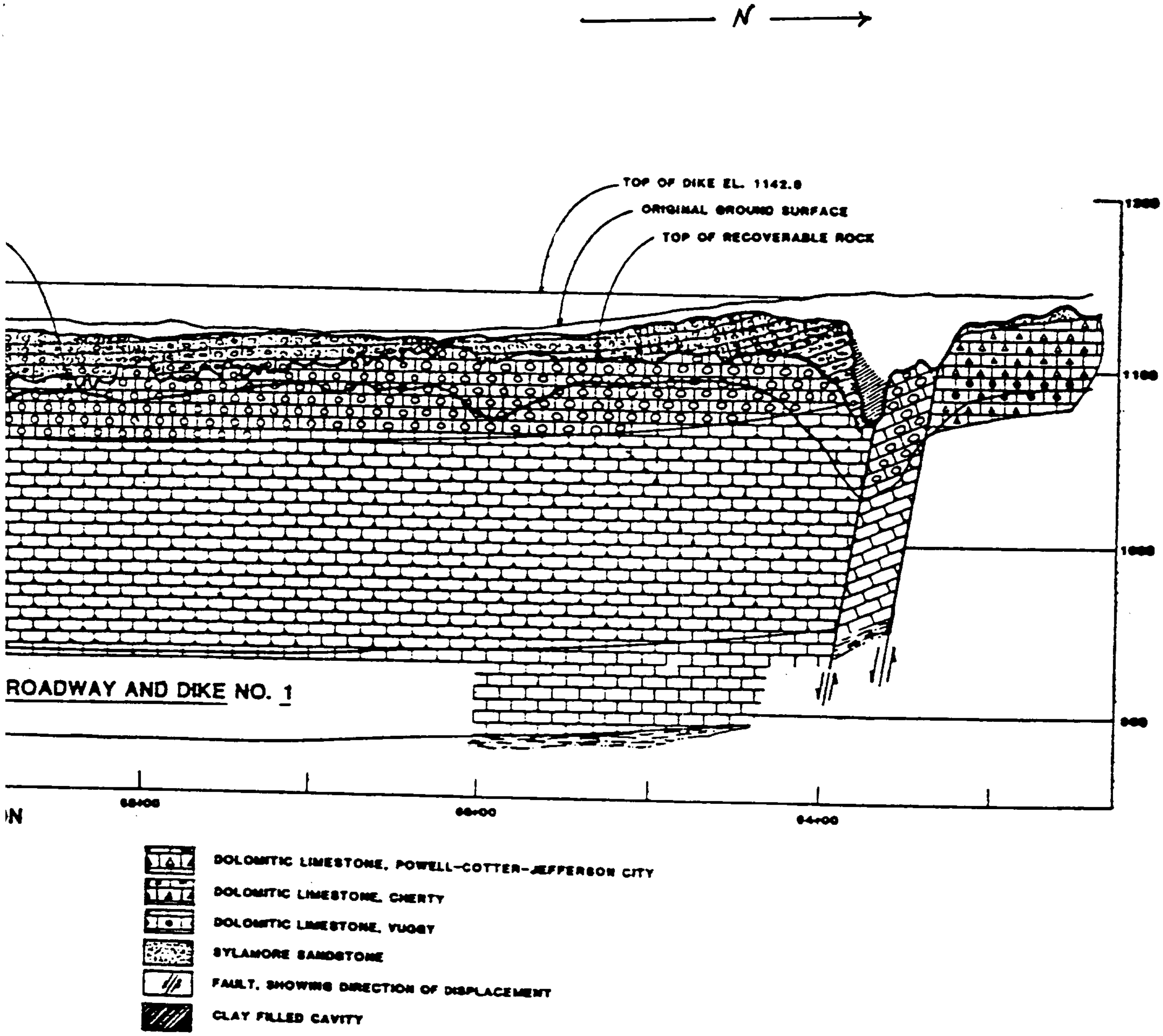


Figure 4. Geologic profil



ɔrofile, Dike 1

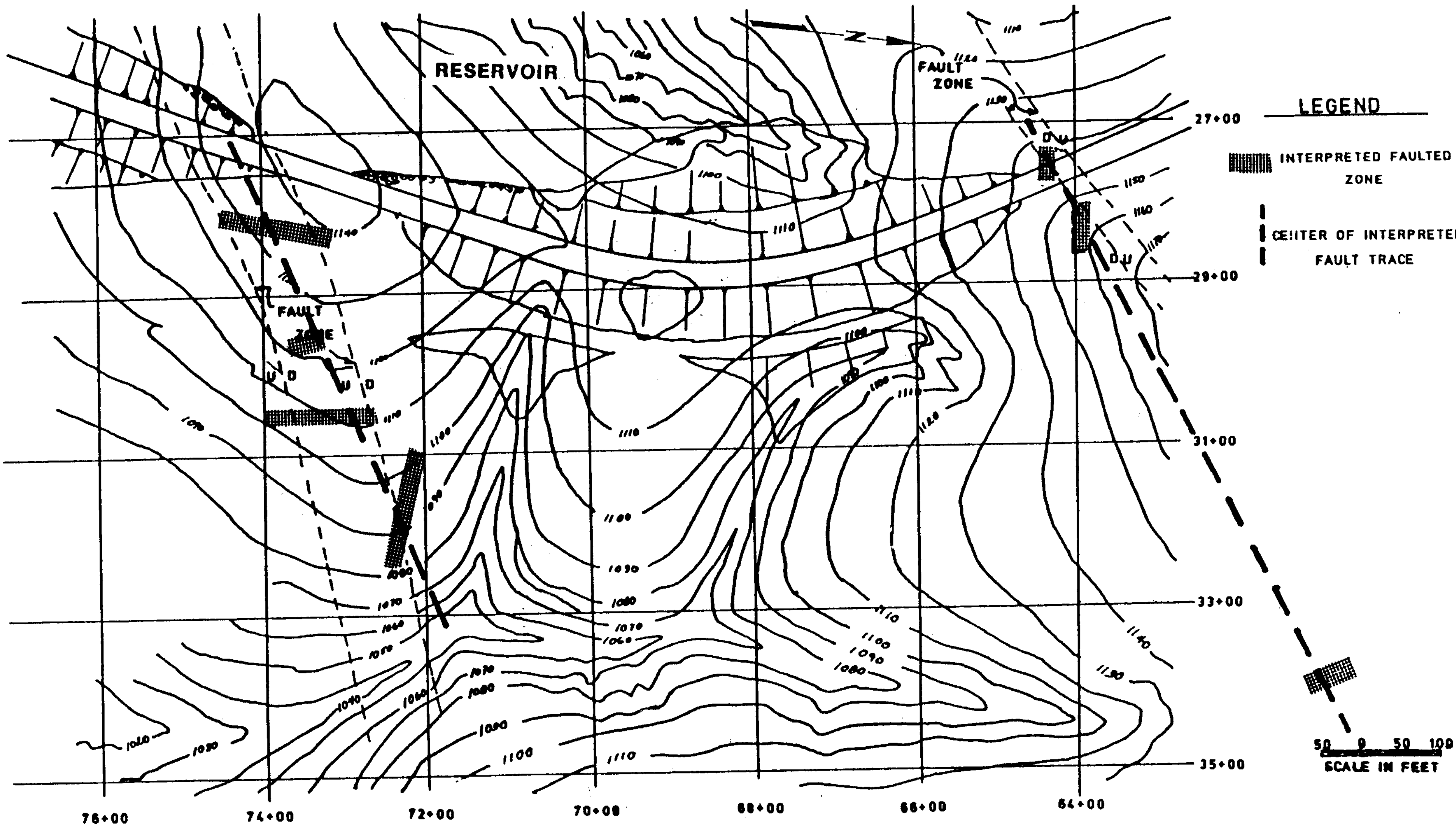


Figure 32. Fault zone locations interpreted from results of geophysical testing

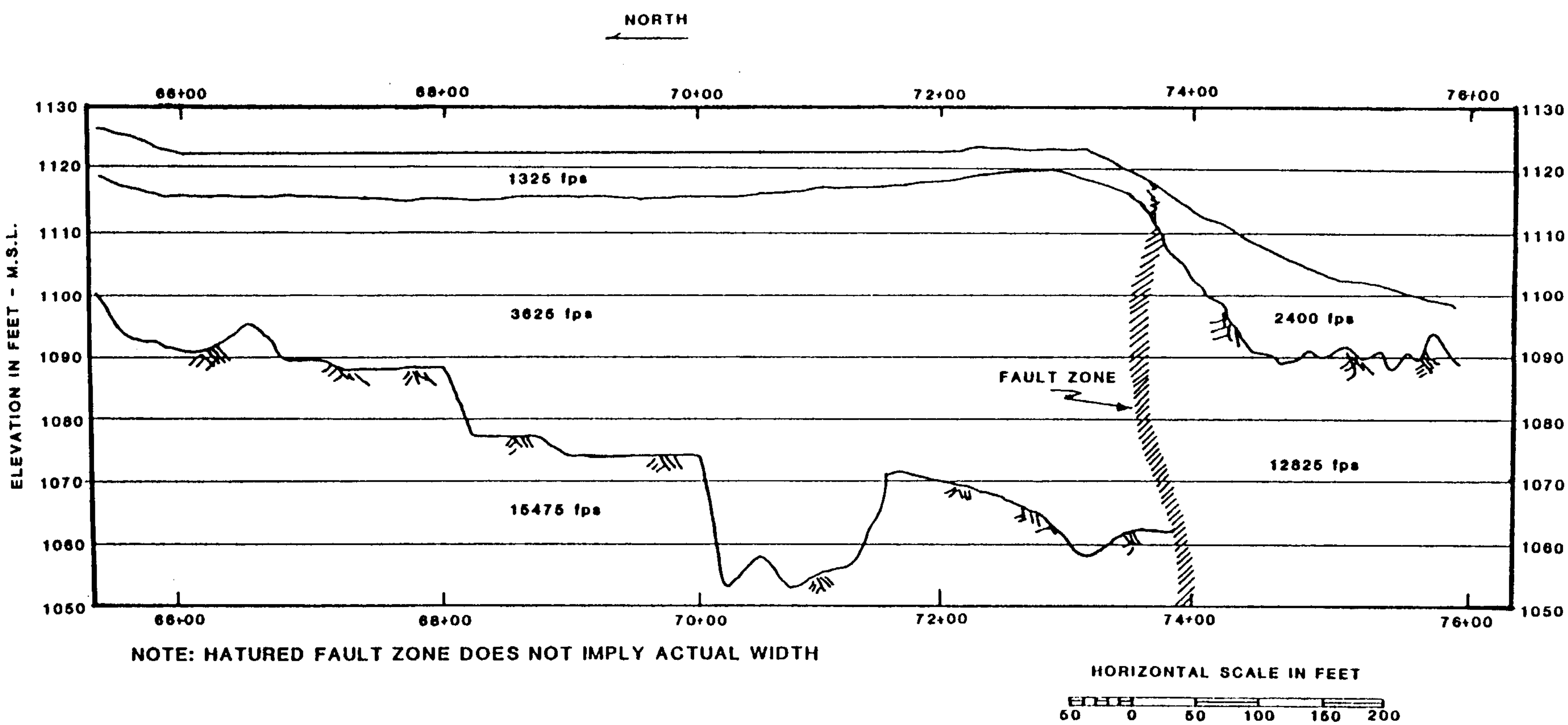


Figure 31. P-wave velocity cross section for Dike 1

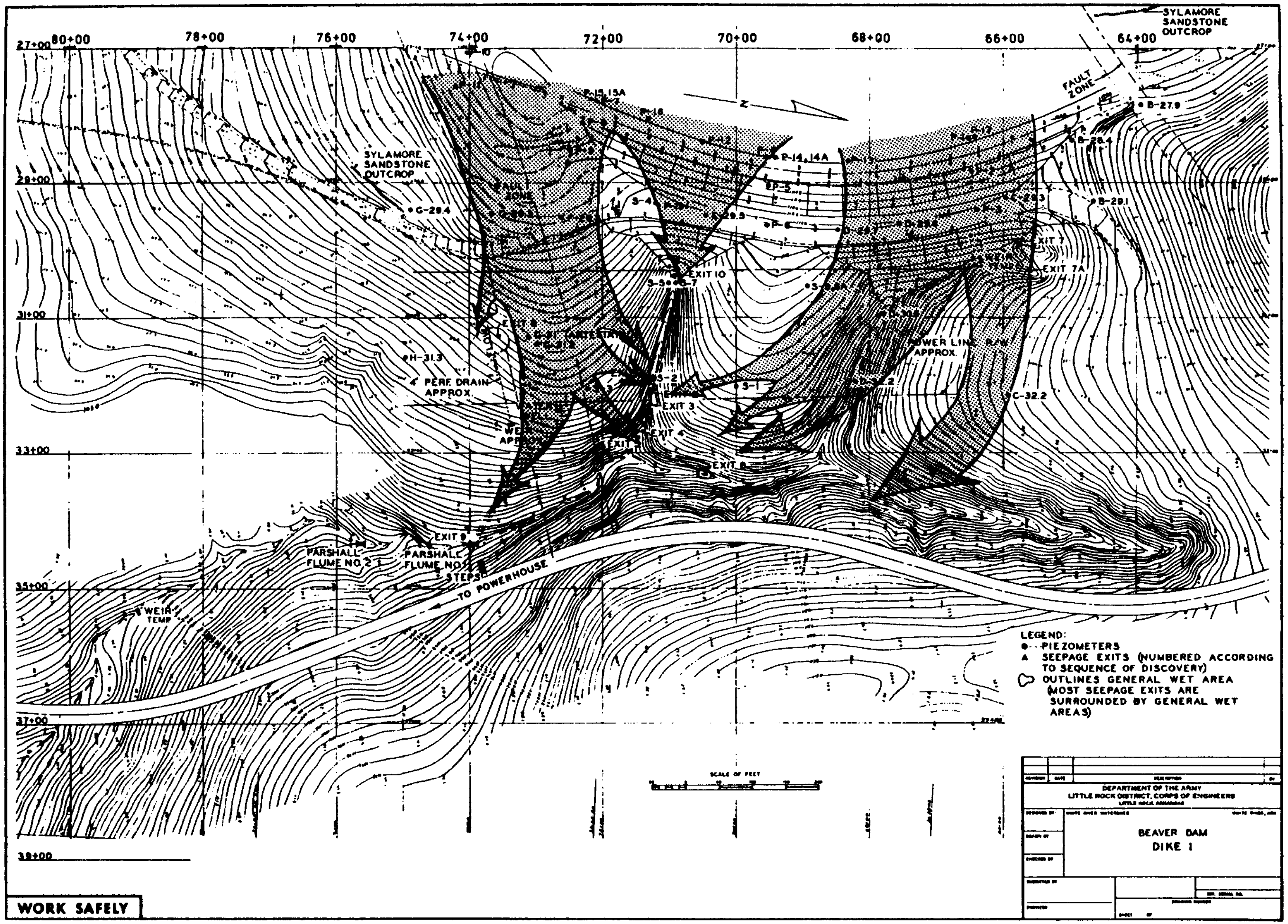
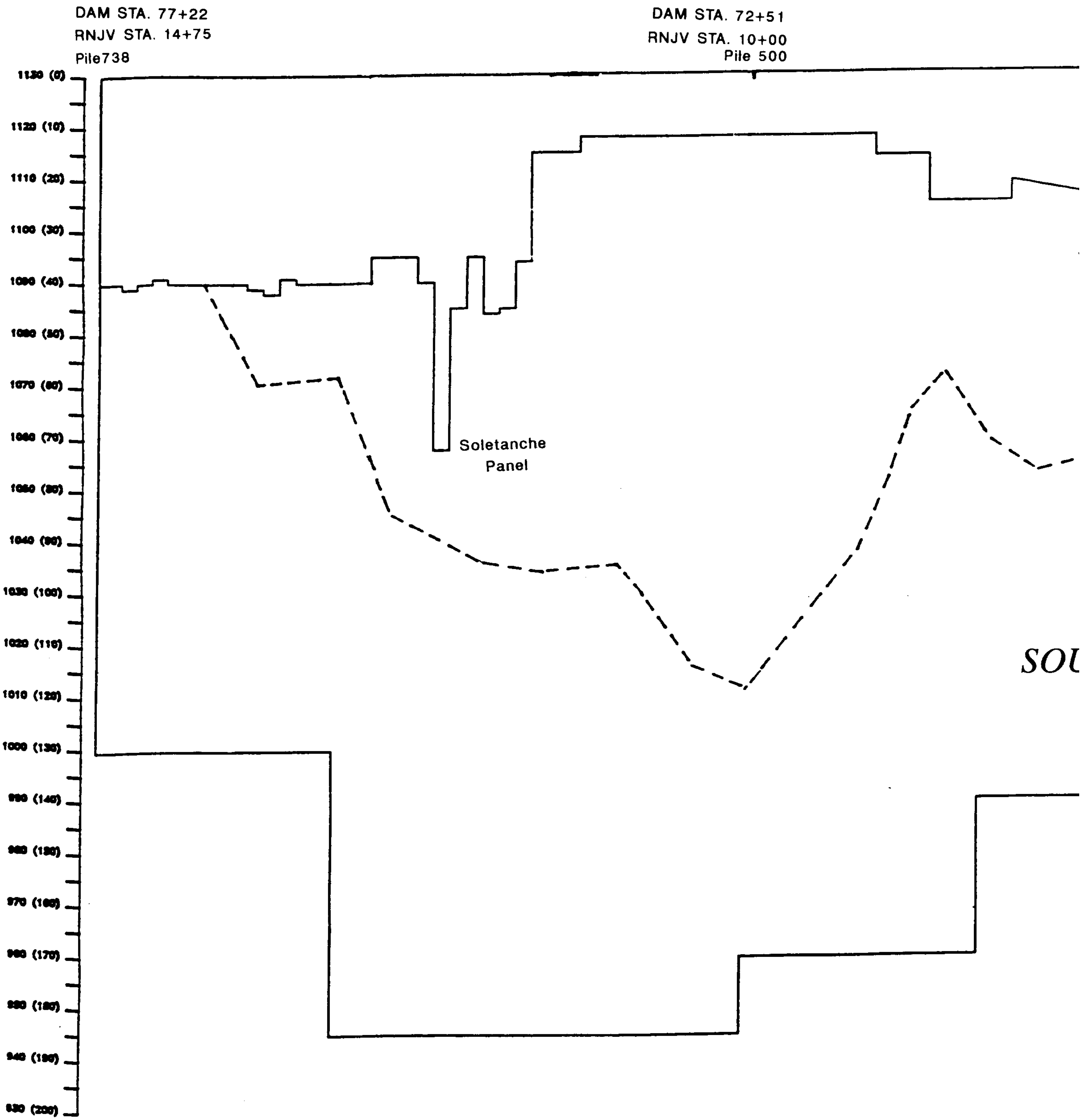
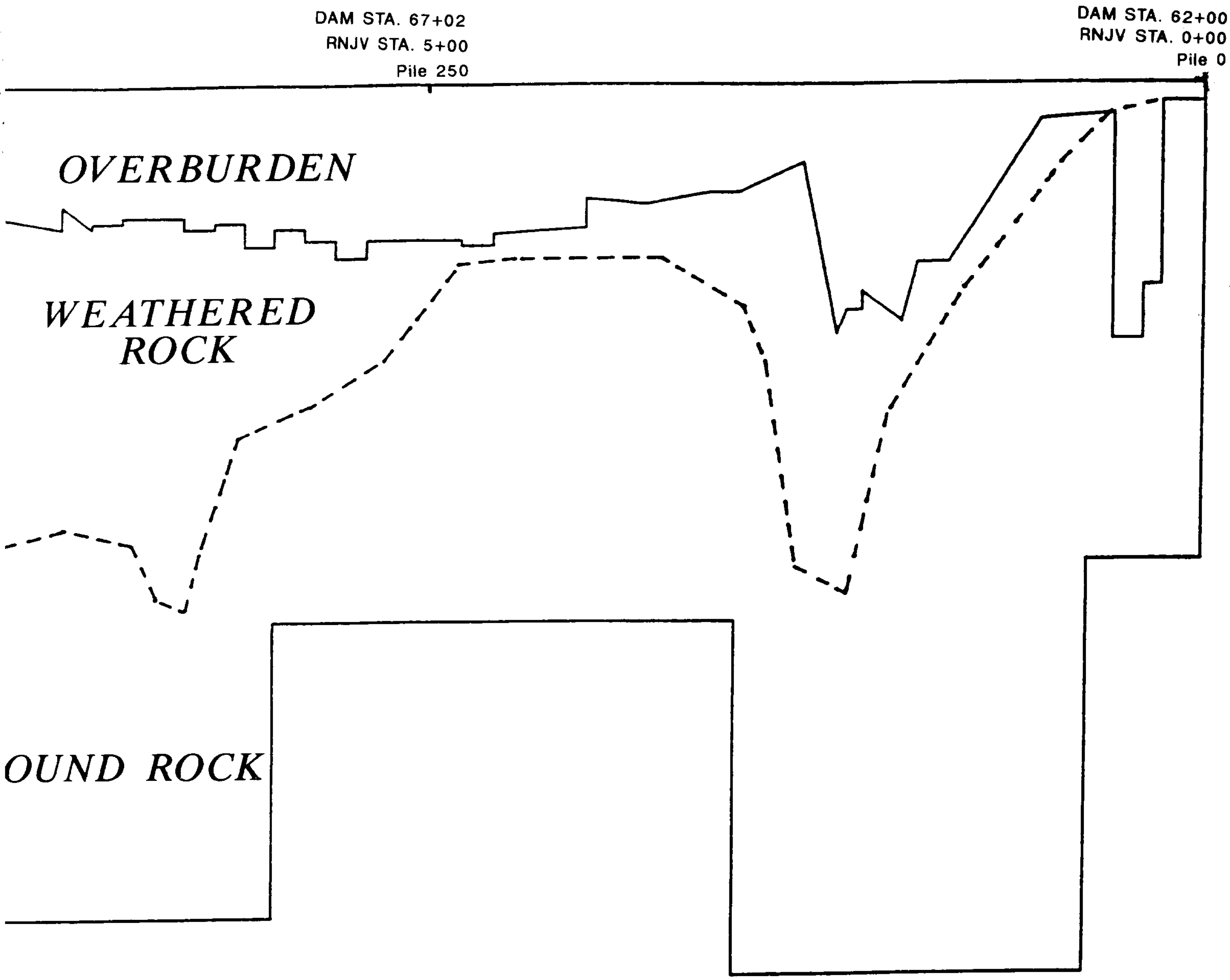


Figure 56. Integrated methods seepage map

BEAVER DAM SECANT P



T PILE CUTOFF WALL

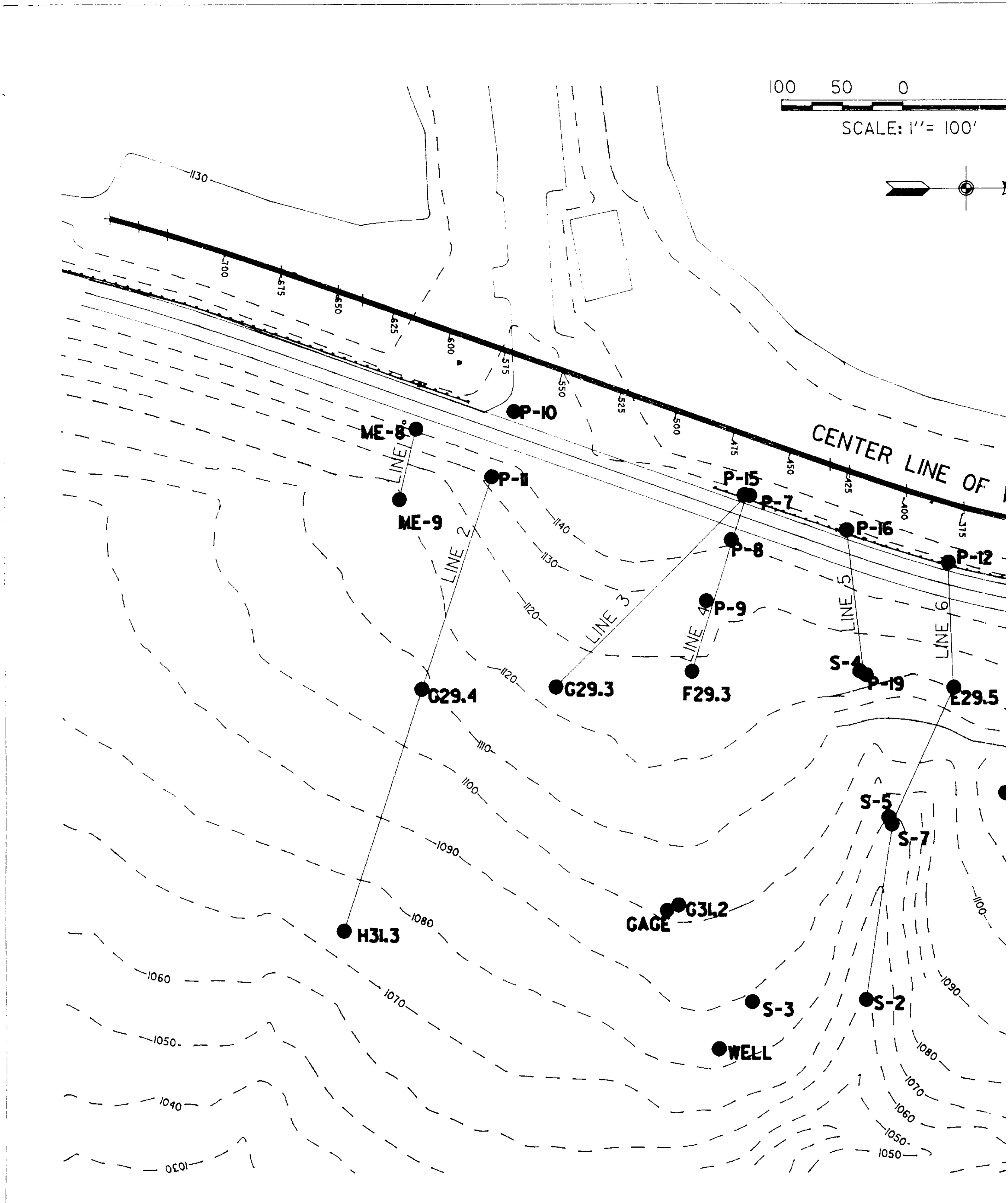


BEAVER DAM COMPLETION REPORT

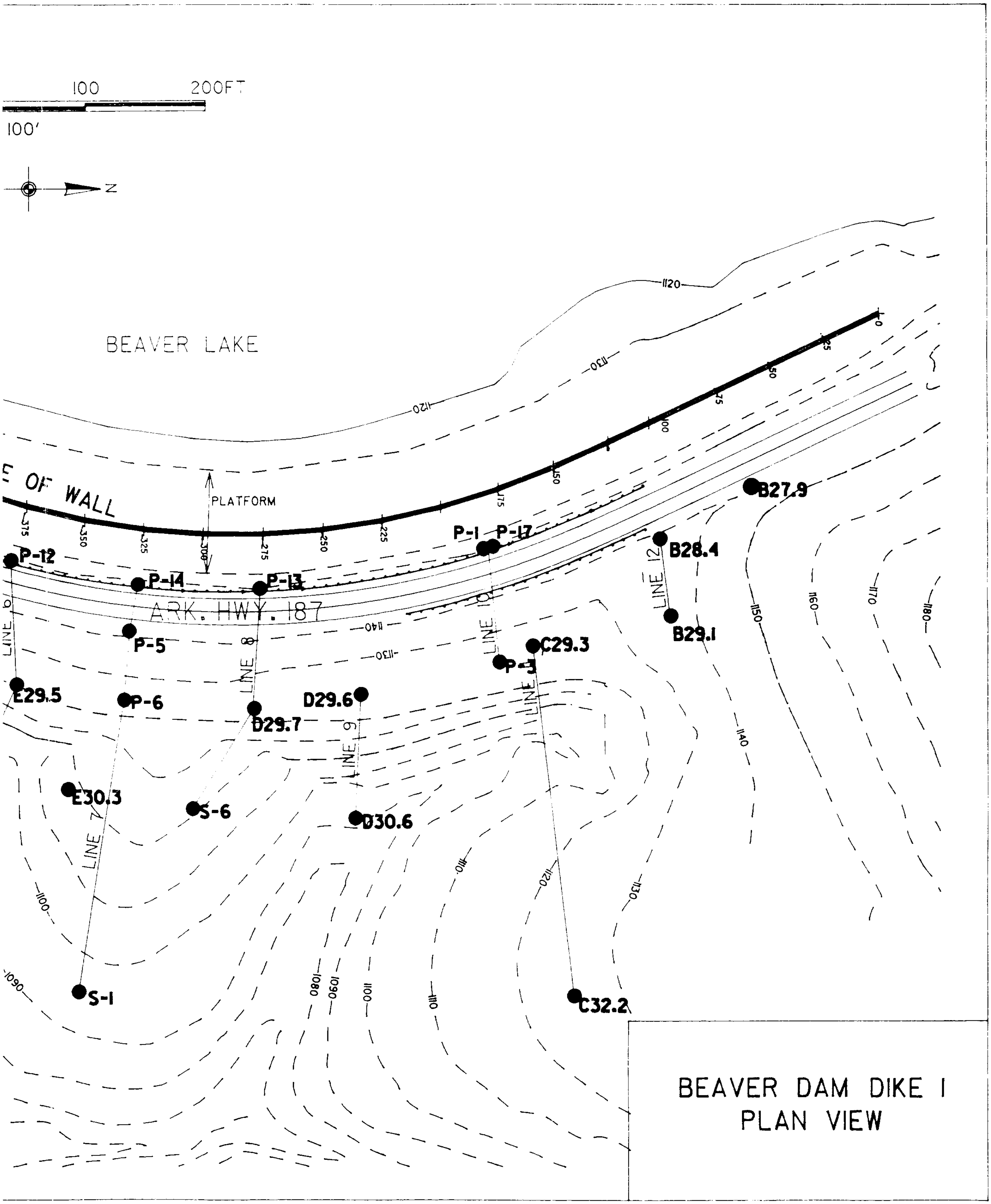
APPENDIX - C

APPENDIX C: INSTRUMENTATION DATA AND DRAWINGS

List of Plates



100 50 0
SCALE: 1''= 100'
CENTER LINE OF
1130
1140
1120
1110
1100
1090
1080
1070
1060
1050
1040
1030
700
675
650
625
600
575
550
525
500
475
450
425
400
375
P-10
ME-8
P-11
ME-9
P-15
P-7
P-8
P-9
P-16
P-12
LINE 1
LINE 2
LINE 3
LINE 4
LINE 5
LINE 6
S-4
P-19
G29.4
G29.3
F29.3
E29.5
S-5
S-7
G3L2
GAGE
H3L3
S-3
S-2
WELL



BEAVER DAM DIKE I
PLAN VIEW

PLATE C-1

PIEZOMETER DATA (Grouped by Rises & Declines)

PZ-No.	Date	PZ Line	Fall (0) Rise (1)	Remarks
				PZ Declines (5/16/93 – 6/28/93)
P-17	05/16/93	10	0	
P-3	05/16/93	10	0	
C29.3A	05/18/93	11	0	Dry in July 1993
C29.3B	05/18/93	11	0	Decline continuing
B27.9B	05/20/93	12	0	
B29.1B	05/20/93	12	0	
B29.1A	05/20/93	12	0	Slow erratic decline, dry in June 1993
B27.9A	05/22/93	12	0	Dry on 7/7/94
B28.4	05/30/93	12	0	Slow decline, falling head test confirm slow reaction.
P-1	06/01/93	10	0	
D29.6A	06/01/93	9	0	Dry in Aug. 1993
C32.2A	06/08/93	11	0	Dry in Aug. 1993
P-13	06/28/93	8	0	Slight decline
				PZ Declines (12/1/93 – 1/1/94)
E29.5A	12/14/93	6	0	Slight decline
D29.7C	01/01/94	8	0	
				PZ Declines (3/15/94 – 4/22/94)
P-11	03/15/94	2	0	
C32.2B	03/17/94	11	0	
P-7	03/18/94	4	0	Became dry
P-8	03/18/94	4	0	Became dry
P-9	03/18/94	4	0	
G29.3B	03/19/94	3	0	
P-16	03/19/94	5	0	
S-4	03/20/94	5	0	Became dry
P-12B	03/21/94	6	0	Noisy until decline, dry in April 1994
G29.3A	03/22/94	3	0	Dry inApril 1994
S-6	03/23/94	8	0	
E29.5A	03/23/94	6	0	Dry in April 1994
P-13	03/23/94	8	0	
P-6	03/24/94	7	0	Became dry
E29.5B	03/24/94	6	0	
P-5	03/24/94	7	0	Became dry
G31.2A	03/24/94	13	0	
S-5	03/25/94	6	0	
D29.7A	03/25/94	8	0	Dry in April 94
P-19	03/26/94	5	0	
S-6A	03/26/94	8	0	
P-17	03/28/94	10	0	
Gage	03/30/94	13	0	
G29.4A	04/01/94	2	0	
D29.7B	04/01/94	8	0	
G29.4B	04/01/94	2	0	
S-7	04/01/94	6	0	
P-12A	04/01/94	6	0	
S-2	04/01/94	6	0	
P-14	04/02/94	7	0	Declined with pool elev.
P-15	04/04/94	3	0	
D29.6B	04/08/94	9	0	
D30.6B	04/20/94	9	0	Slight decline

PIEZOMETER DATA (Grouped by Rises & Declines)

PZ-No.	Date	PZ Line	Fall (0) Rise (1)	Remarks
B29.1B	04/20/94	12	0	
B27.9B	04/22/94	12	0	Became noisy in Nov. 1994
				PZ Declines (8/1/94 – 8/19/94)
Well	08/01/94	13	0	Significant (200') decrease after 8/1/94
D32.2A	08/19/94	11	0	Slight decline
				PZ Rise (6/12/93)
P-14	06/12/93	7	1	Returned to normal on 6/23/93
				PZ Rise (11/15/93)
B27.9B	11/15/93	12	1	Slight rise 11/15/93 to 4/20/94.
				PZ Rises (2/24/94 – 2/26/94)
P-17	02/24/94	10	1	
B29.1B	02/26/94	12	1	Rise 2/26/94 to 4/15/94.
				PZ Rise (9/15/94)
P-14	09/15/94	7	1	Rise due to falling head test, remained elevated after.
				PZ's That Follow Lake Pool El.
P-11	01/01/93	2	2	Follows pool until 3/15/94
S-1	01/01/93	7	2	Follows pool El.
Gage	01/01/93	13	2	Noisy with pool until 3/30/94 decline, now dry
G29.3B	01/01/93	3	2	Follows pool until 3/19/94
G29.3A	01/01/93	3	2	Follows pool until 3/22/94
				PZ's w/o Change through Contract
S-3	01/01/93	13	3	No significant changes
P-6	01/01/93	7	3	No significant changes
D32.2B	01/01/93	9	3	No significant changes
				Main Embankment PZ's Adjacent to Dike-1
ME-7A		1	3	No significant changes
ME-7B		1	0	No significant changes
ME-8A		1	0	Dropped 3/15/94 in sinc. with dike 1 pz's
ME-8B		1	0	Being tested 3/94 until 5/94; dropped during this time

(2) PZ follows pool el.
(3) No significant changes in pz during contract

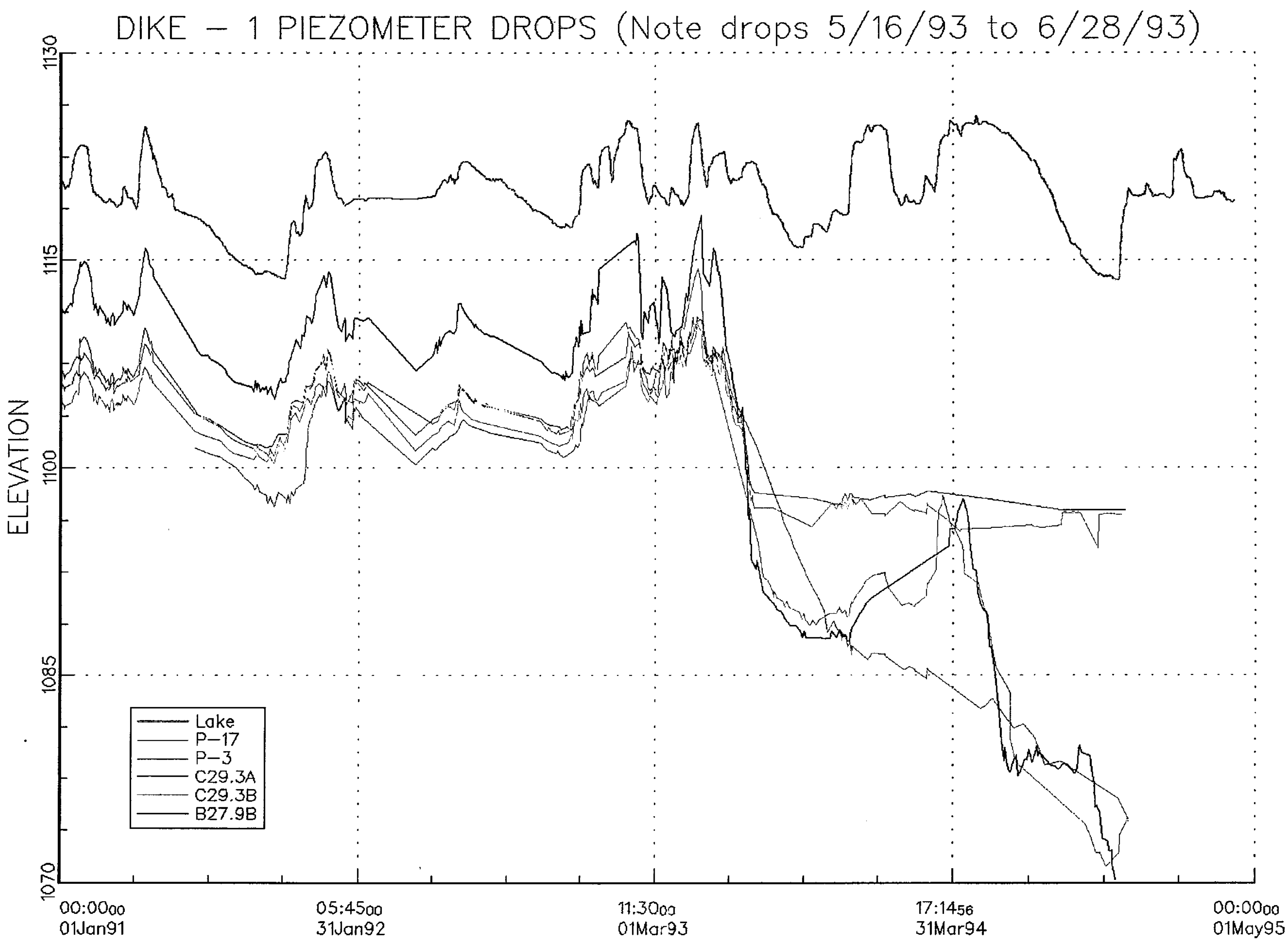

DIKE – 1 PIEZOMETER DROPS (Note drops 5/16/93 to 6/28/93)
ELEVATION
1130
1115
1100
1085
1070
00:00:00
01Jan91
05:45:00
31Jan92
11:30:00
01Mar93
17:14:56
31Mar94
00:00:00
01May95
Lake
P–17
P–3
C29.3A
C29.3B
B27.9B

PLATE C-3

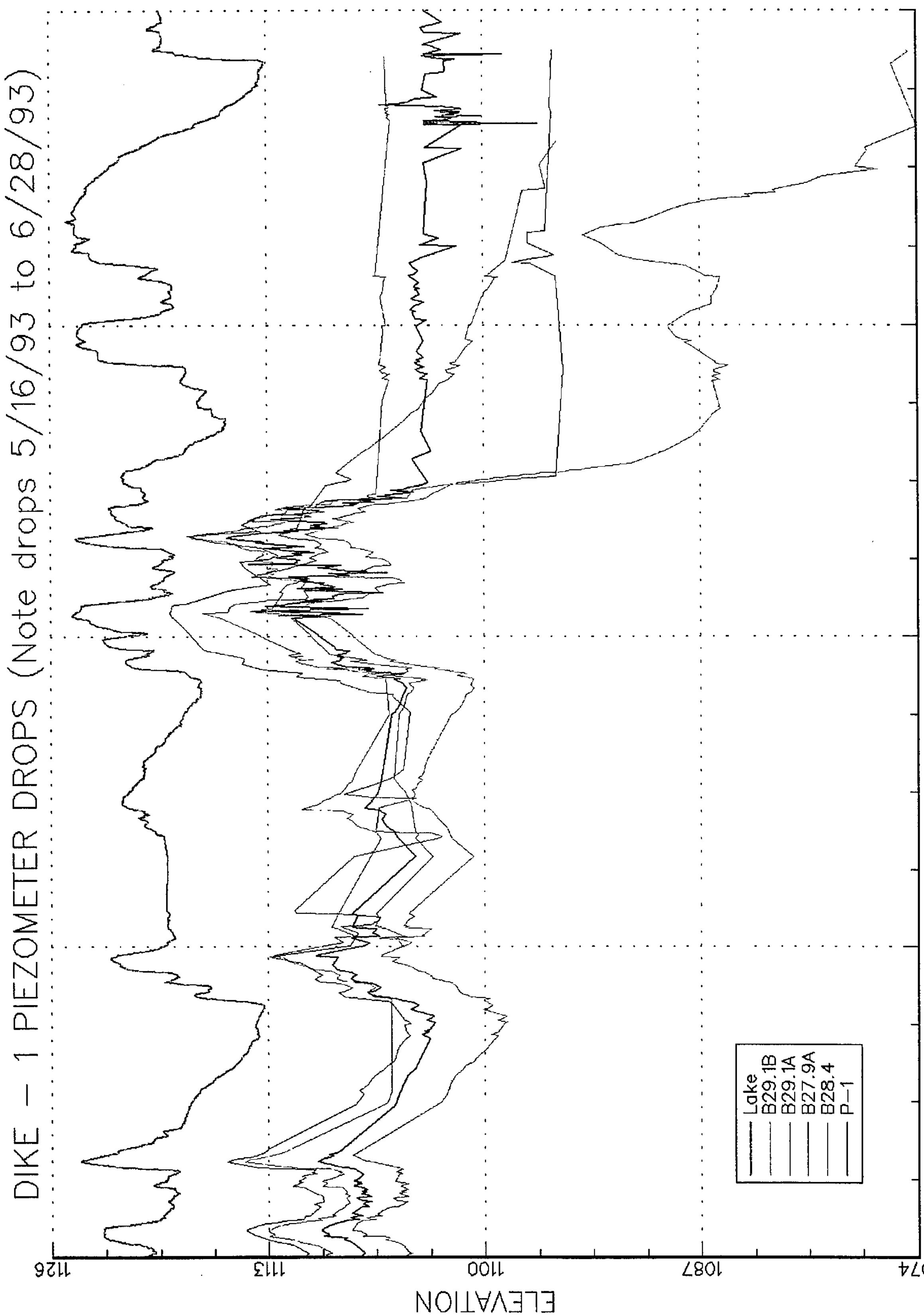

DIKE — 1 PIEZOMETER DROPS (Note drops 5/16/93 to 6/28/93)
ELEVATION
1126
1113
1100
1087
1074
00:00oo 01Jan91
06:00oo 01Jan92
12:00oo 31Dec92
18:00oo 31Dec93
00:00oo 01Jan95
Lake
B29.1B
B29.1A
B27.9A
B28.4
P-1

PLATE C-4

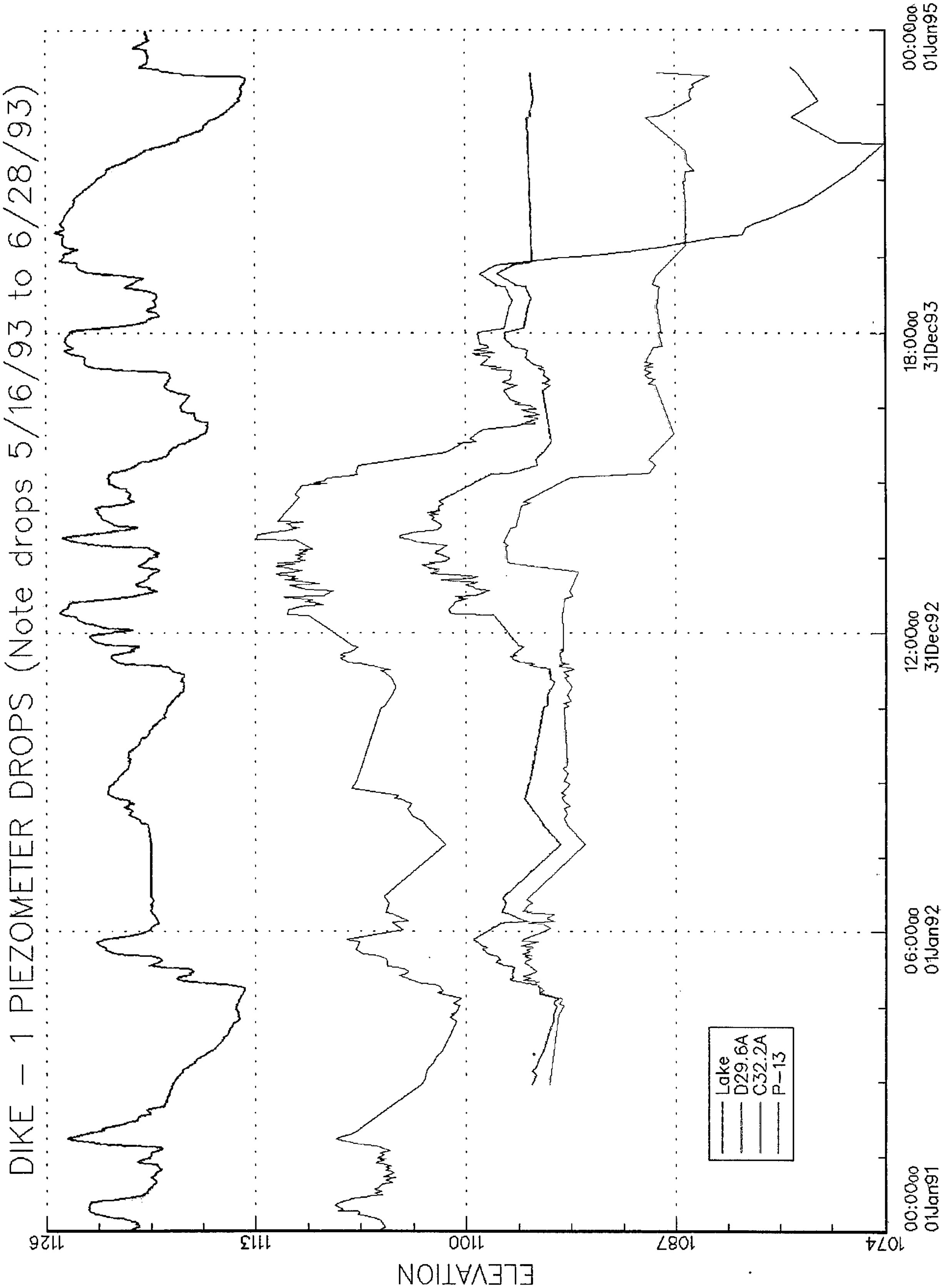
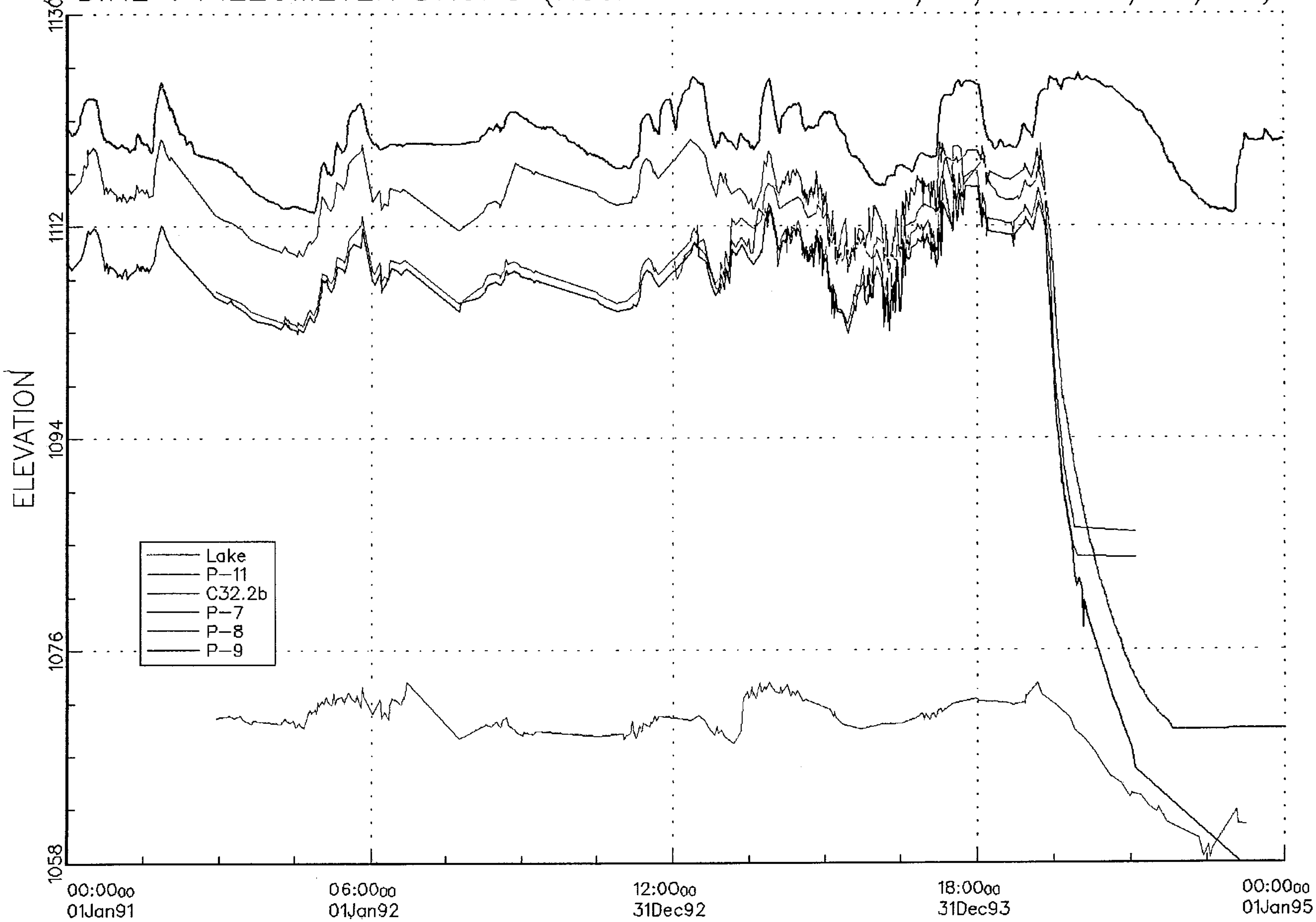
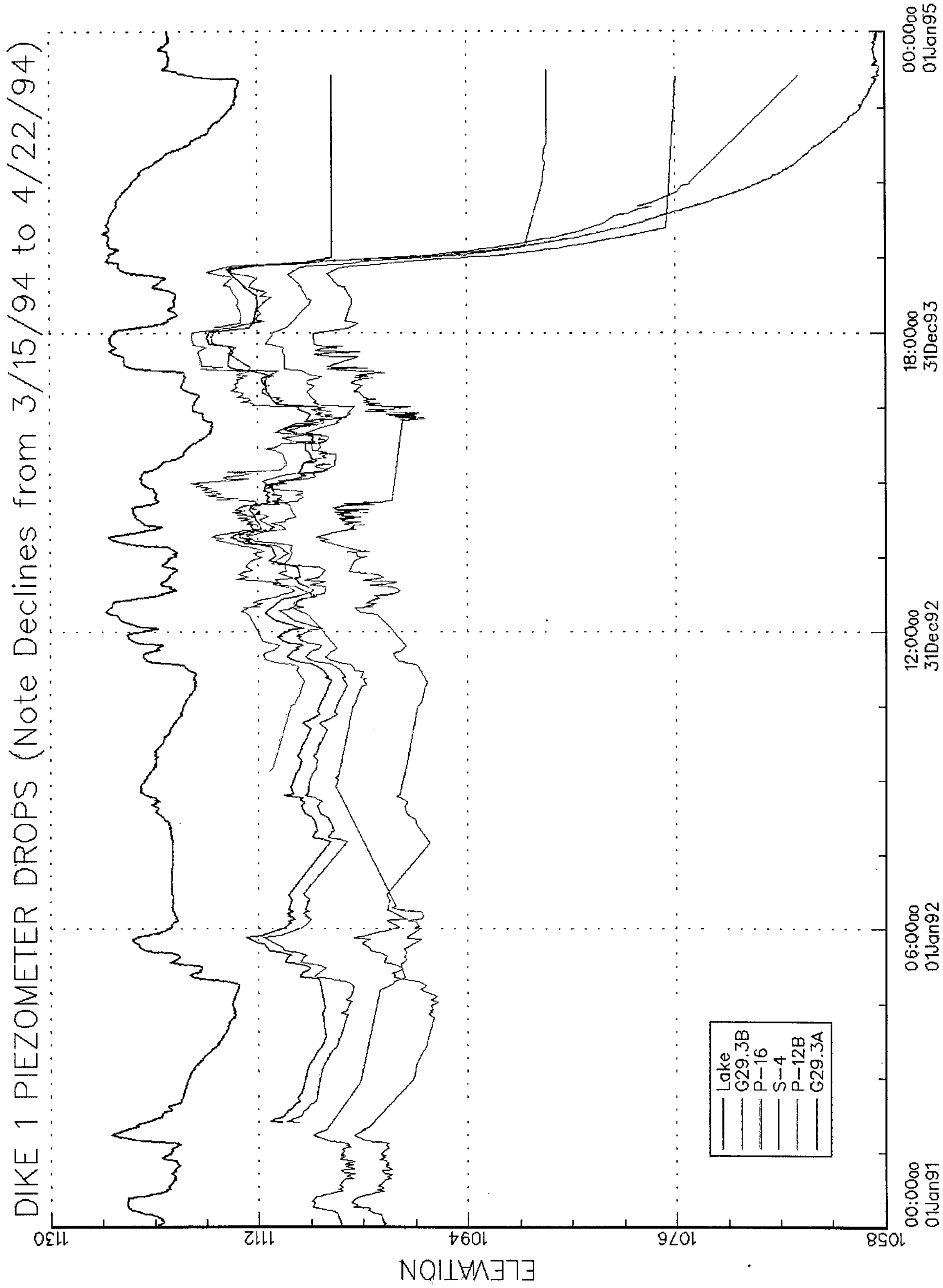
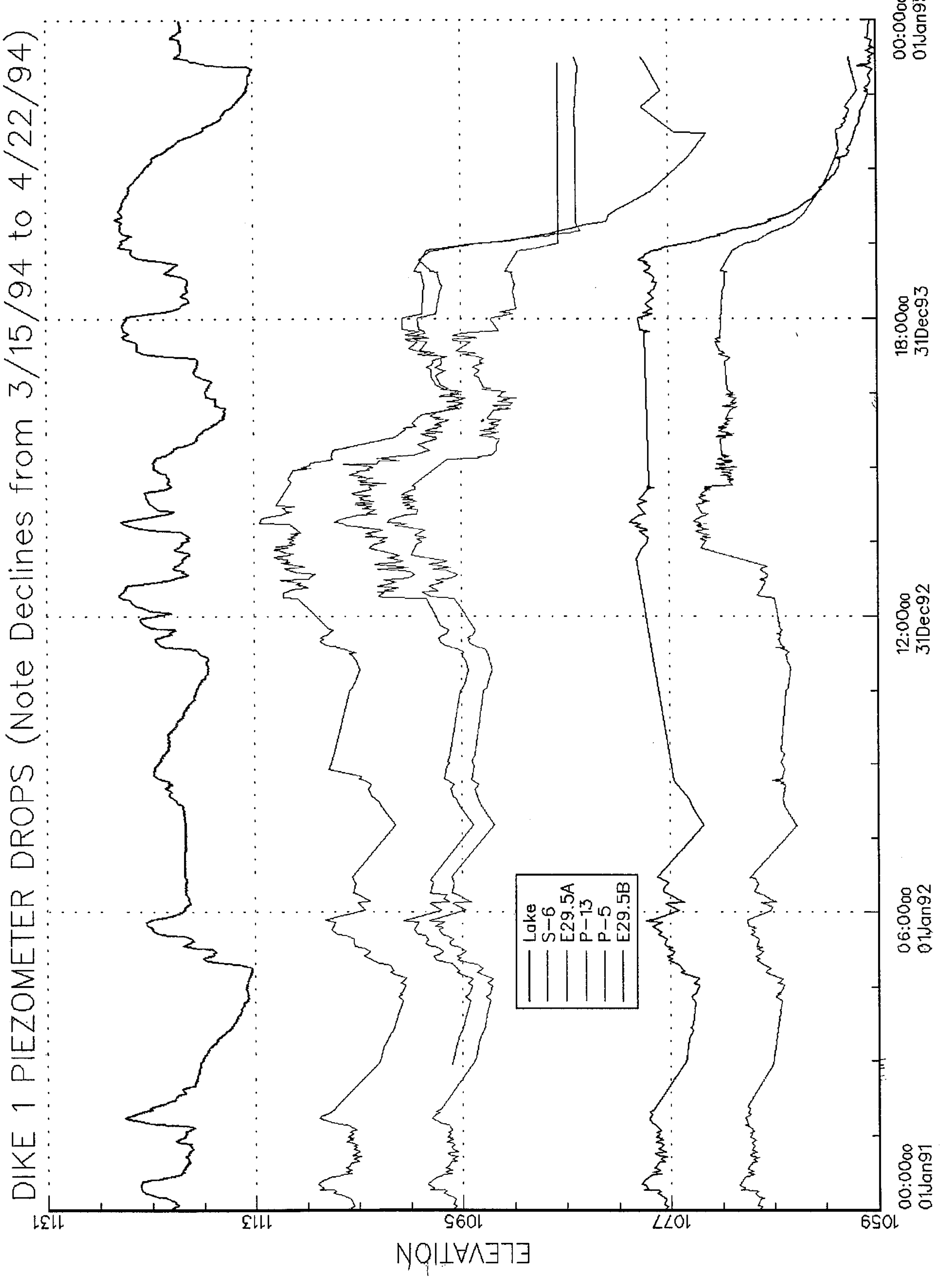

DIKE — 1 PIEZOMETER DROPS (Note drops 5/16/93 to 6/28/93)
ELEVATION
1126
1113
1100
1087
1074
00:00:00 01Jan91
06:00:00 01Jan92
12:00:00 31Dec92
18:00:00 31Dec93
00:00:00 01Jan95
Lake
D29.6A
C32.2A
P-13

PLATE C-5

DIKE 1 PIEZOMETER DROPS (Note Declines from 3/15/94 to 4/22/94)




DIKE 1 PIEZOMETER DROPS (Note Declines from 3/15/94 to 4/22/94)
ELEVATION
1130
1112
1094
1076
1058
00:00:00 01Jan91
06:00:00 01Jan92
12:00:00 31Dec92
18:00:00 31Dec93
00:00:00 01Jan95
Lake
G29.3B
P-16
S-4
P-12B
G29.3A



DIKE 1 PIEZOMETER DROPS (Note Declines from 3/15/94 to 4/22/94)
ELEVATION
1131
1113
1095
1077
1059
00:00oo 01Jan91
06:00oo 01Jan92
12:00oo 31Dec92
18:00oo 31Dec93
00:00oo 01Jan95
Lake
S-6
E29.5A
P-13
P-5
E29.5B

DIKE 1 PIEZOMETER DROPS (Note Declines from 3/15/94 to 4/22/94)

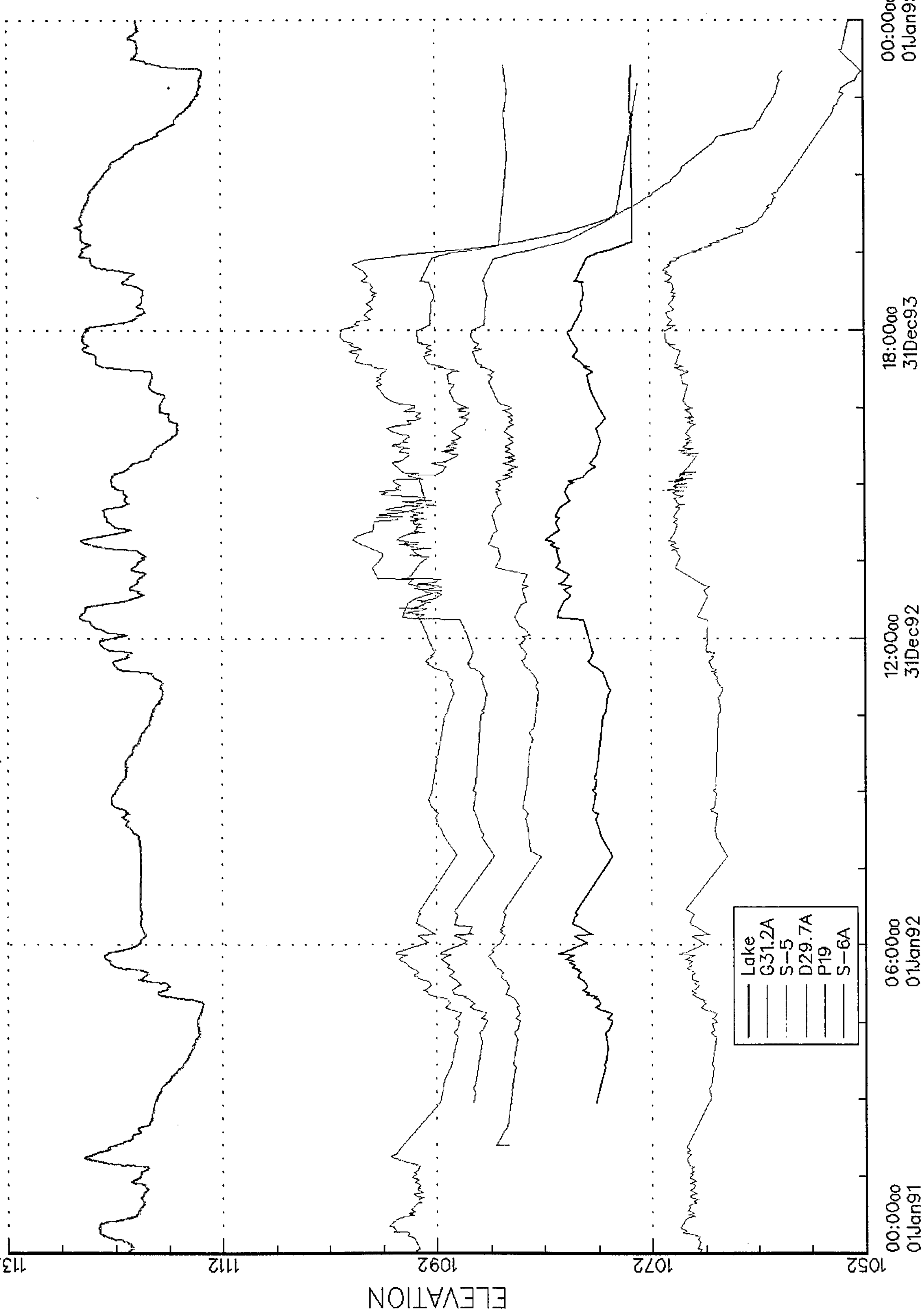
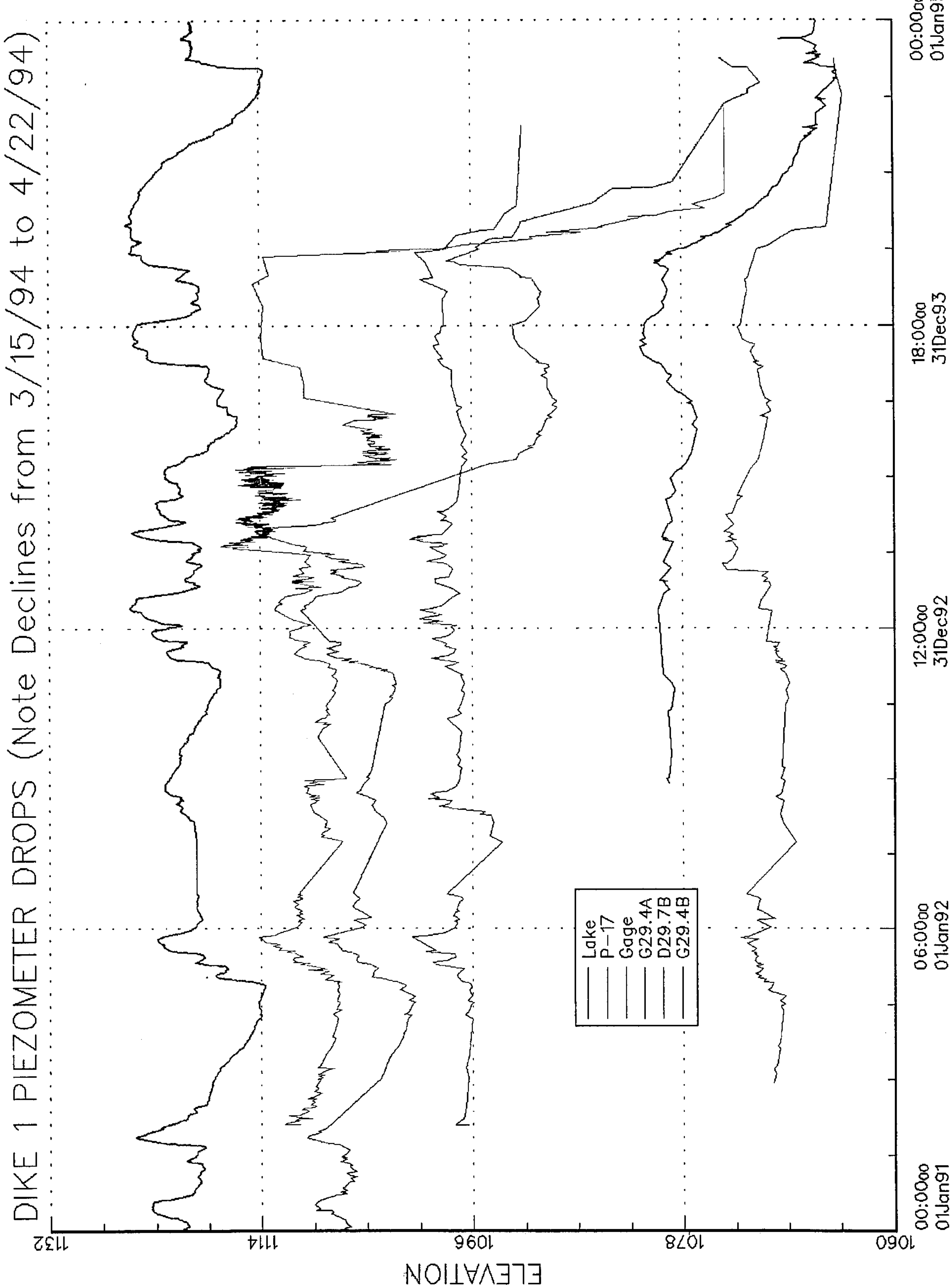
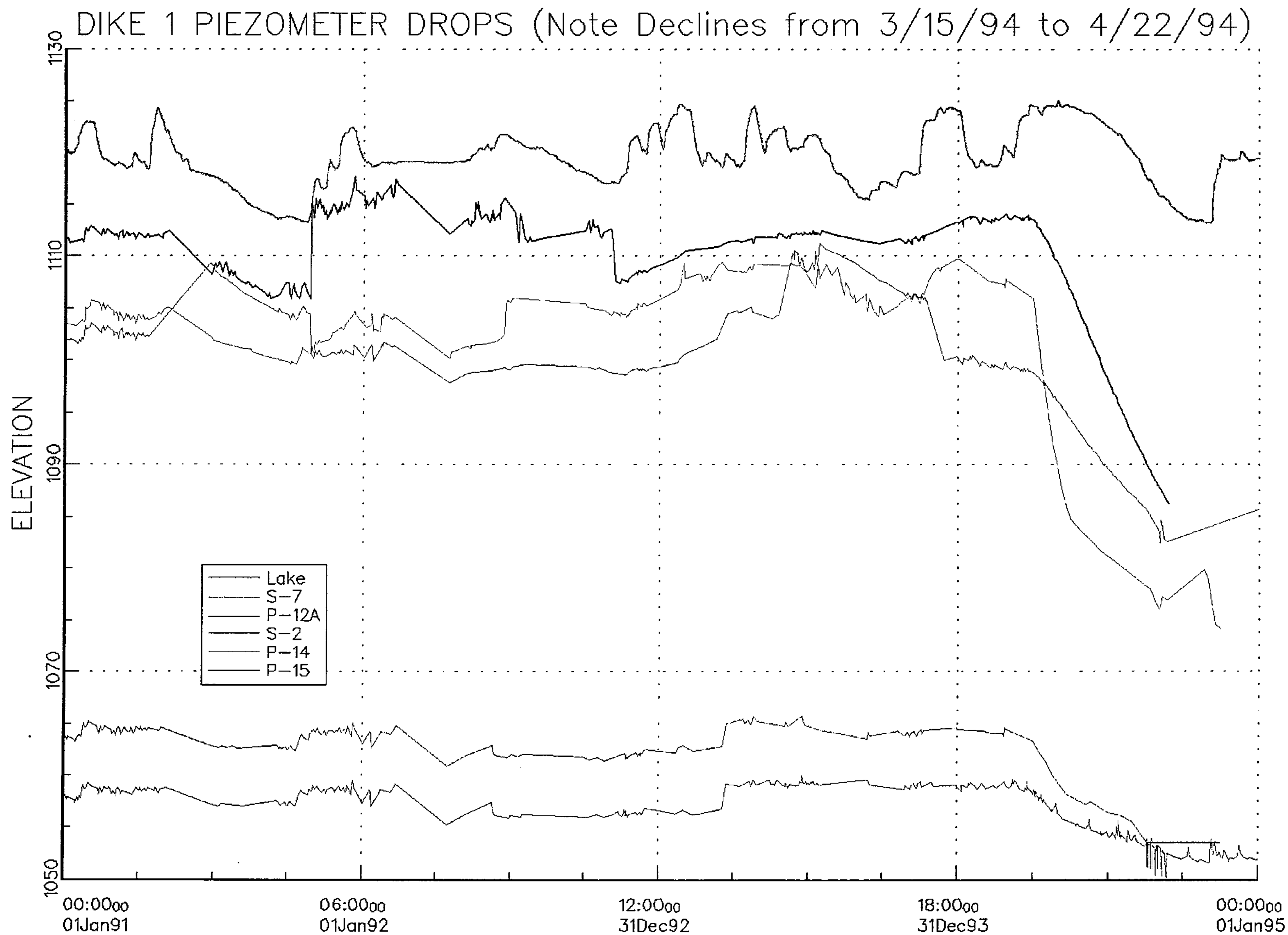


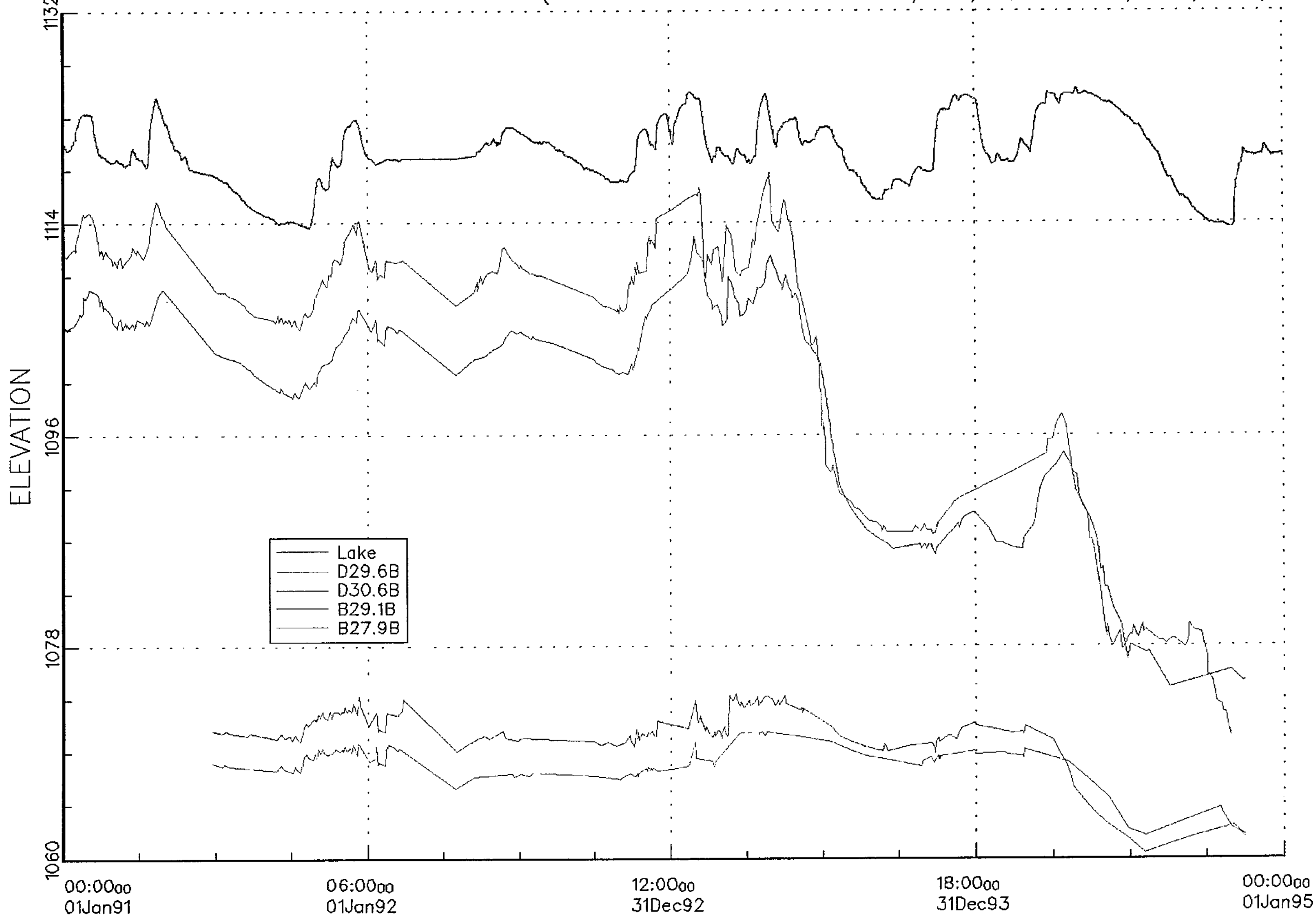
PLATE C-9



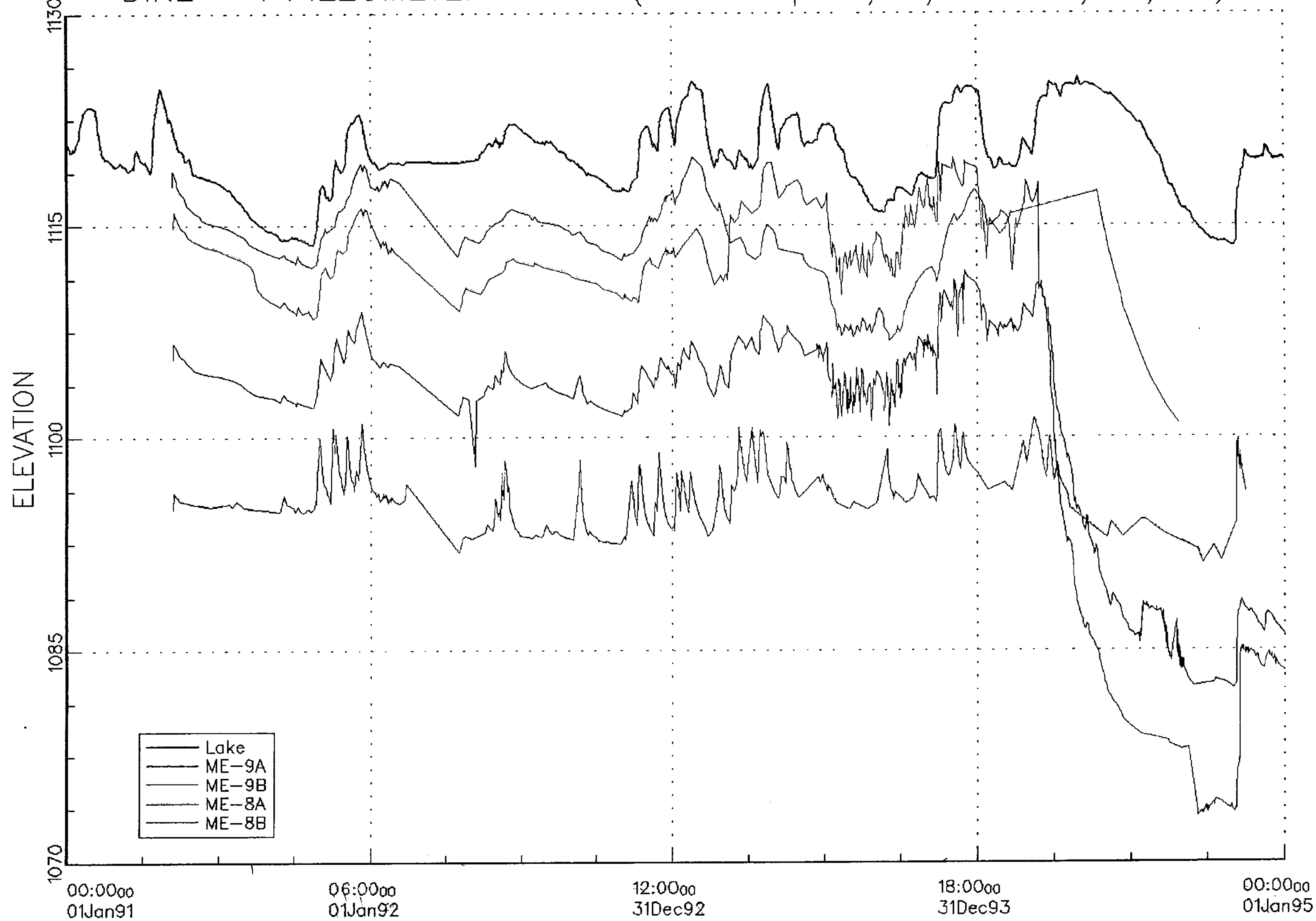
DIKE 1 PIEZOMETER DROPS (Note Declines from 3/15/94 to 4/22/94)
ELEVATION
1132
1114
1096
1078
1060
00:00oo 01Jan91
06:00oo 01Jan92
12:00oo 31Dec92
18:00oo 31Dec93
00:00oo 01Jan95
Lake
P-17
Gage
G29.4A
D29.7B
G29.4B


DIKE 1 PIEZOMETER DROPS (Note Declines from 3/15/94 to 4/22/94)
ELEVATION
1130
1110
1090
1070
1050
00:00oo
01Jan91
06:00oo
01Jan92
12:00oo
31Dec92
18:00oo
31Dec93
00:00oo
01Jan95
Lake
S-7
P-12A
S-2
P-14
P-15

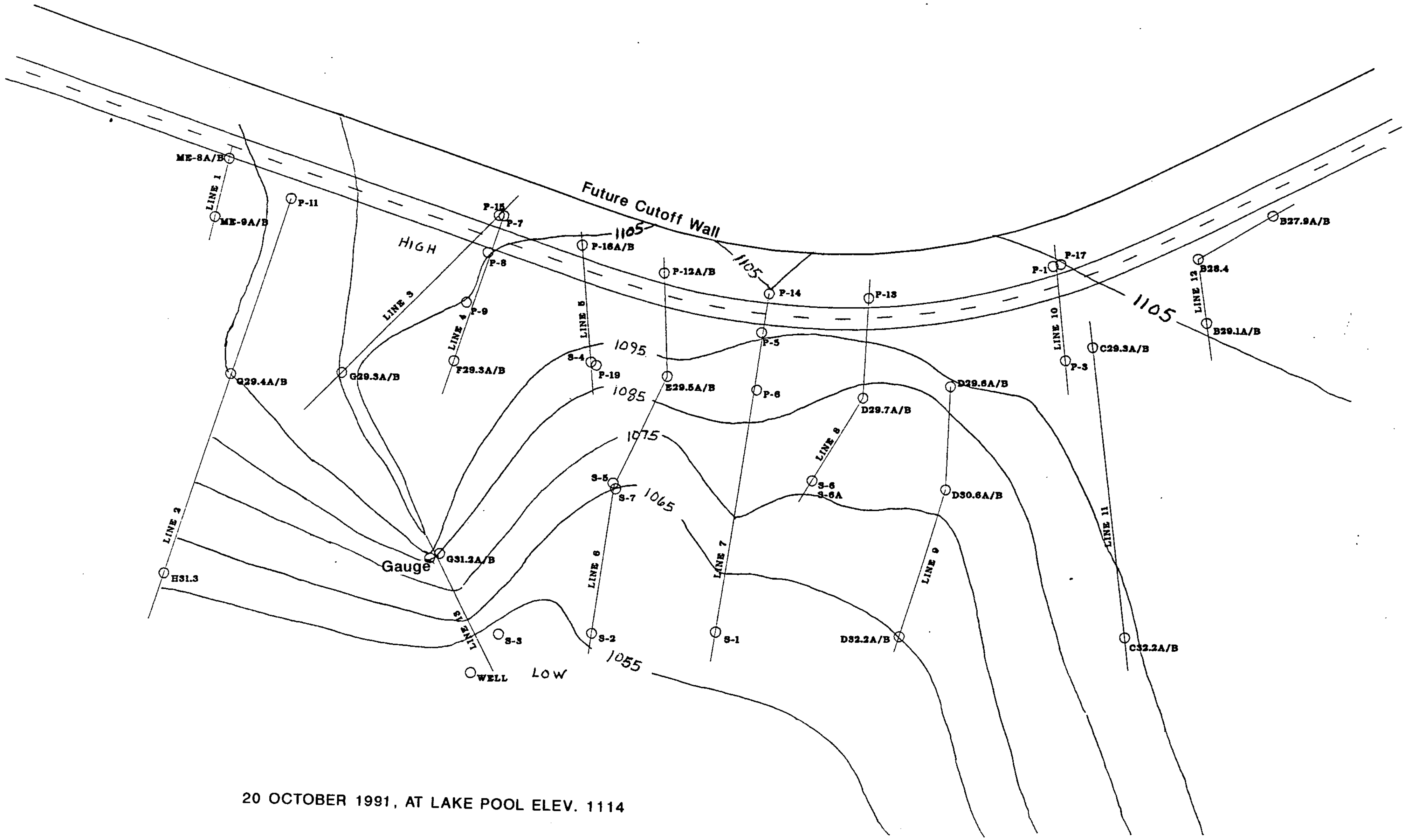
DIKE 1 PIEZOMETER DROPS (Note Declines from 3/15/94 to 4/22/94)



DIKE – 1 PIEZOMETER DROPS (Note drops 5/16/93 to 6/28/93)

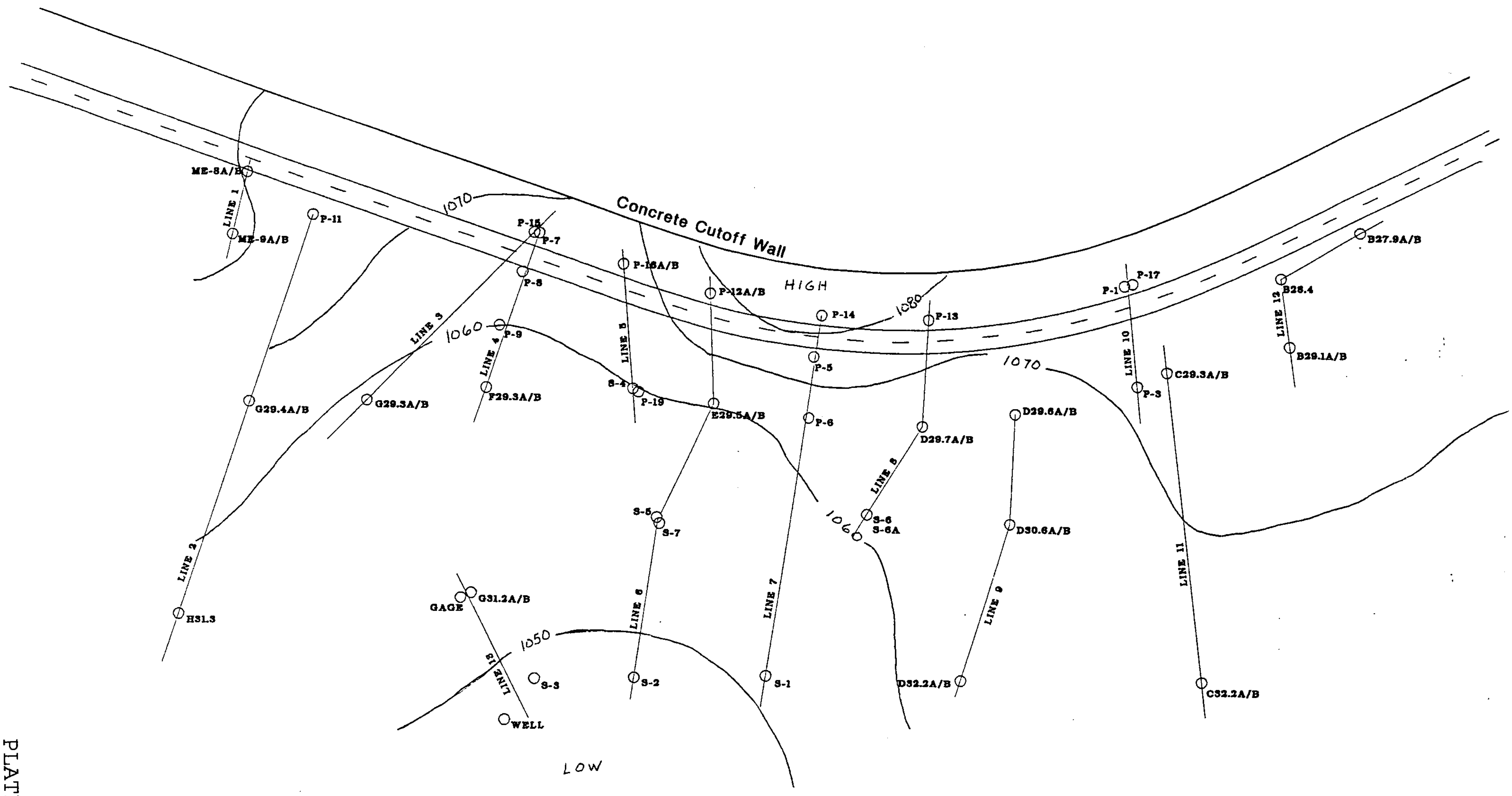


PIEZOMETRIC CONTOUR MAP OF DIKE-1



20 OCTOBER 1991, AT LAKE POOL ELEV. 1114

PIEZOMETRIC CONTOUR MAP OF DIKE-1



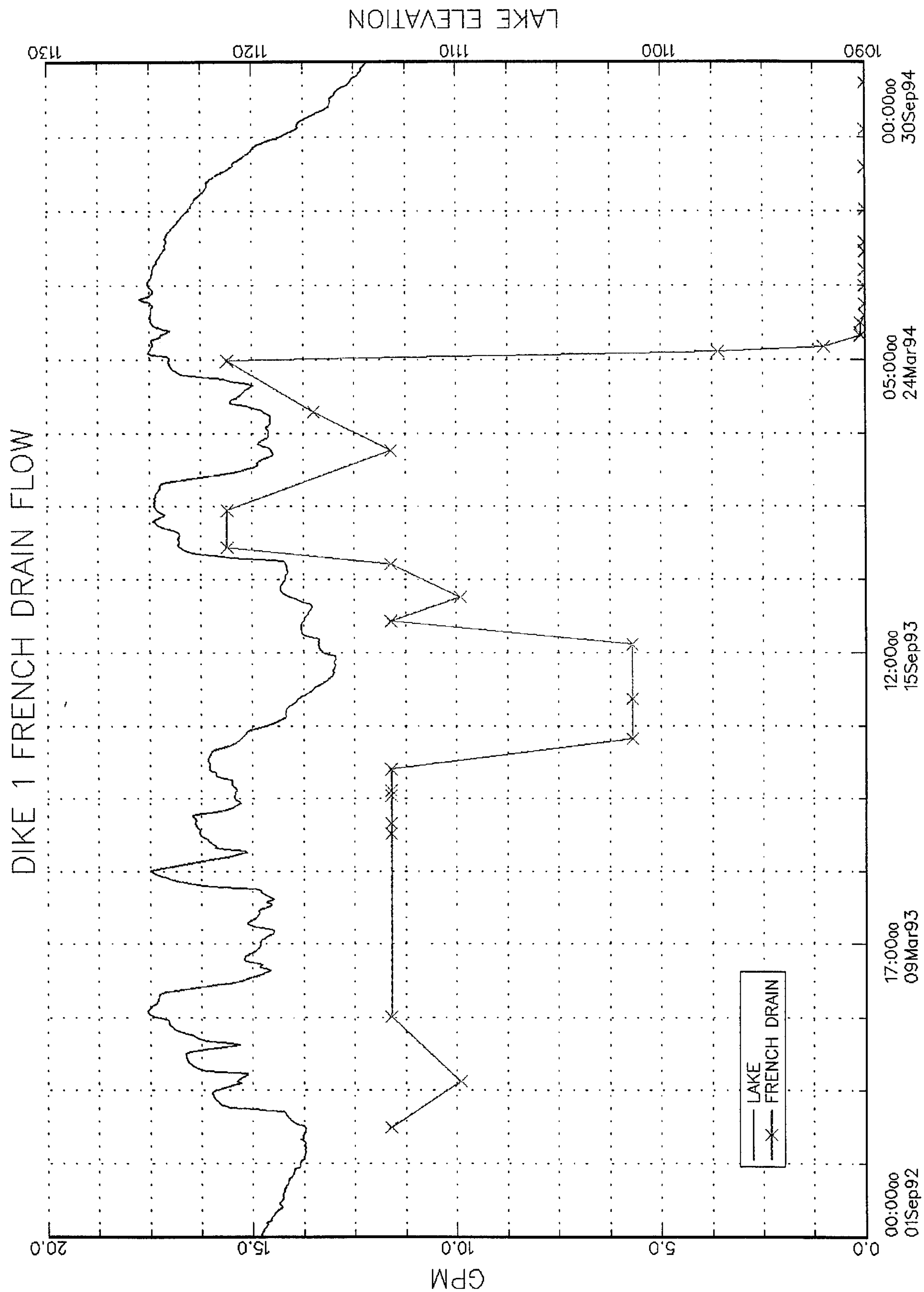
20 OCT 1994, AT LAKE POOL ELEV. 1114

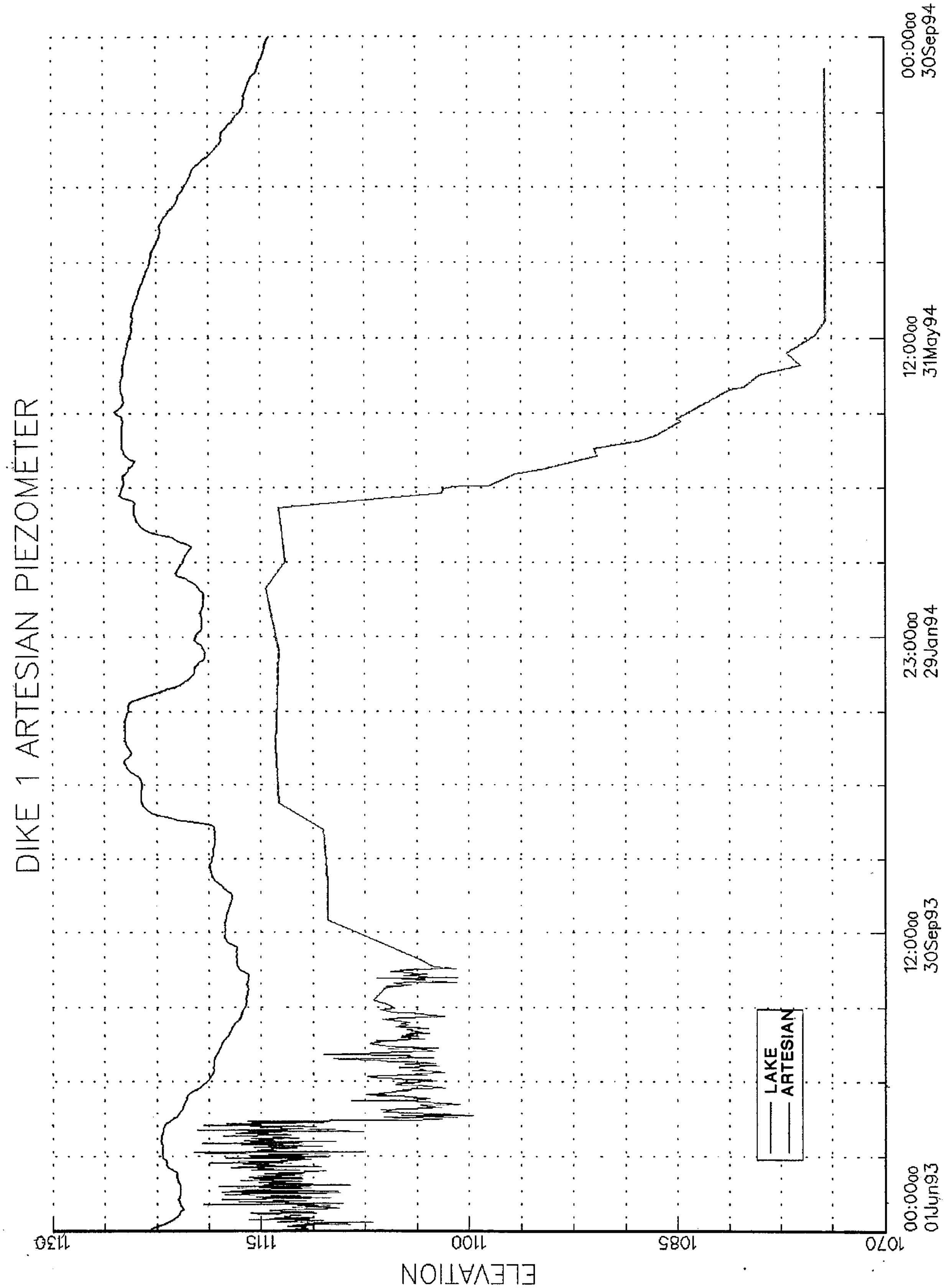
SEEPAGE DATA

DATE	AREA-1 1070 – 1020	AREA-2 1100 – 1076	AREA-3 1098	AREA-4 1104	AREA-5 1046	FLUME-1 GPM 1030	SANDBOIL 1024	FRENCH DRAIN (GPM) 1076	ARTESIAN WELL (EL.) 1104	SOUTH RAVINE WEIR (GPM) 1042	REMARKS (Elevations)
10/22/90	Clear	Clear	Clear	Clear	Clear	204.5	Clear	11.6			None
11/10/92	Clear	Clear	Clear	Clear	Clear	179.2	Clear	11.6	TOG at art. well = EL 1104		Cavities encountered in piles 82,&102
12/10/92	Clear	Clear	Clear	Clear	Clear	272.8	Clear	9.9			Drilling pile 24
01/21/93	Clear	Clear	Clear	Clear	Clear	204.5	Clear	11.6			Drilling piles 136 &336
04/15/93	Clear	Clear	Clear	Clear	Clear	379.1	v. active	32.0			Heavy Rains; Drilling piles 296 & 129
05/20/93	Clear	Clear	Clear	Clear	Clear	258.6	v. active	11.6			White streak in French Drain Area Drilling piles 117 & 275
05/27/93	Clear	Clear	Clear	Clear	Clear	258.6	Clear	11.6			White streak in French Drain Area
06/08/93	Clear	Clear	Clear	Dry	Clear	N.R.	Clear	N.R.			Area – 4 dry today
06/14/93	Clear	Clear	Clear	Almost Dry	Clear	231.0	Clear	11.6			None
06/17/93	Clear	Clear	Clear	Almost Dry	Clear	462.1	Clear	11.6			Contractor began pumping waste water into S. ravine 6–19–93
06/30/93	Clear	Clear	Clear	Almost Dry	Clear	462.1	Clear	32.0			Rained previous 2 days; No contractor waste water discharge
07/01/93	Clear	Clear	Clear	Dry	Clear	462.1	Clear	11.6			Contractor pumping into N. ravine
07/21/93	Clear	Clear	Clear	Dry	Clear	316.9	Clear	5.7			Seepage in Area 1 seems to have increased
08/16/93	Clear	Clear	Clear	Dry	Clear	231.0	Clear	5.7			Contractor pumping waste water
09/20/93	Clear	Clear	Clear	Dry	Clear	445.1	Clear	5.7			Contractor pumping waste water
10/05/93	Clear	Clear	Clear	Dry	Clear	217.7	Clear	11.6	1110.2	180.5	V Notch wier installed in S. Ravine
10/21/93	Clear	Clear	Clear	Dry	Clear	272.8	Clear	9.9	1110.2	180.5	Rained today
11/11/93	Clear	Clear	5 sec. to fill 1 pint	Dry	Clear	244.7	Clear	11.6	1110.5	192.5	None
11/22/93	Clear	Clear	4 sec. to fill 1 pint	Dry	Clear	258.6	v. active	15.6	1113.7	192.5	Lake elev. 1124
12/16/93	Clear	Clear	4 sec. to fill 1 pint	Dry	Clear	258.6	v. active	15.6	1113.9	184.4	Lake elev. 1124
01/24/94	Clear	Clear	same	Dry	Clear	231.0	v. active	11.6	1113.7	200.7	Contractor not pumping waste water Lake el. 1118

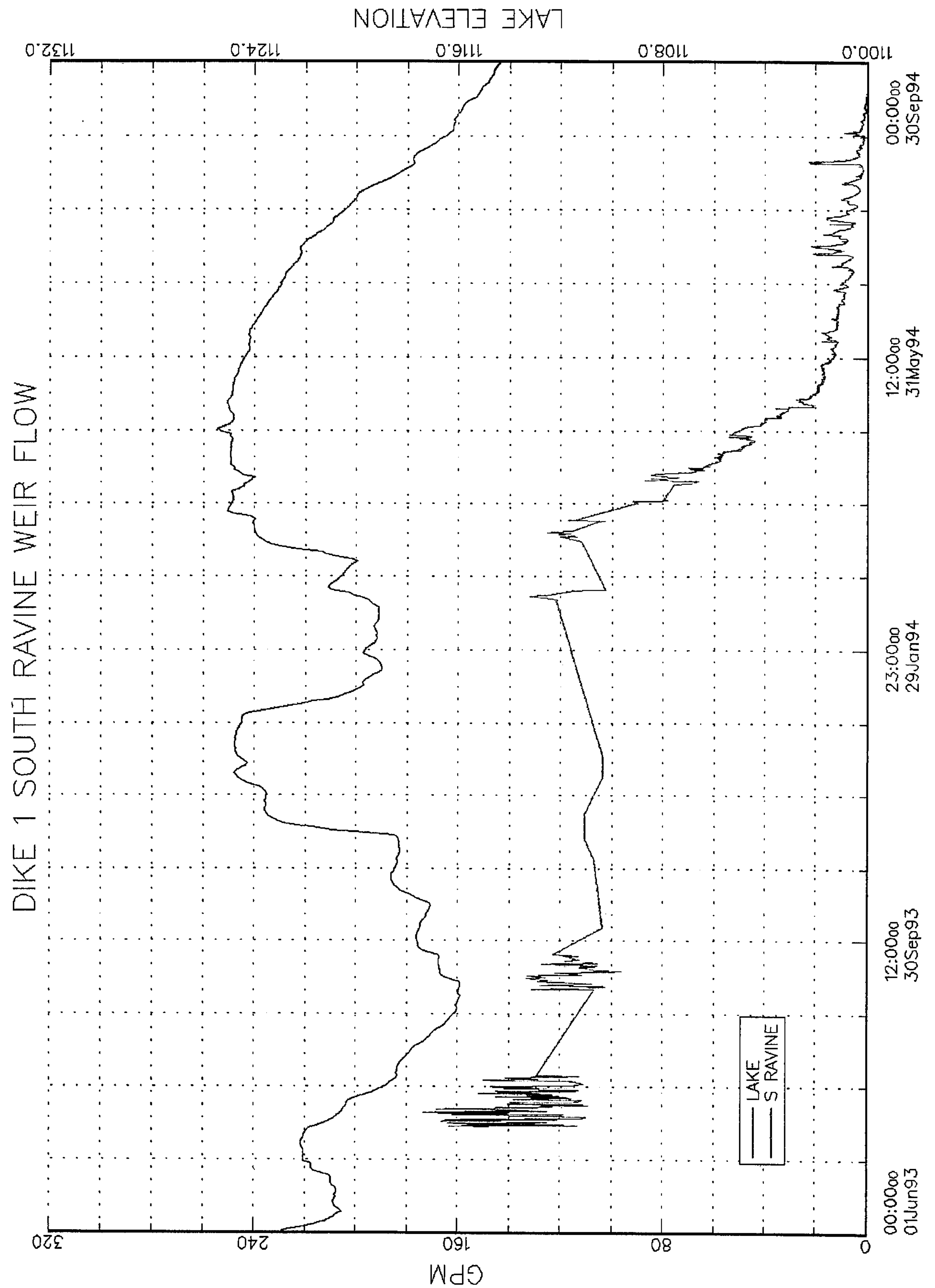
SEEPAGE DATA

DATE	AREA-1 1070 - 1020	AREA-2 1100 - 1076	AREA-3 1098	AREA-4 1104	AREA-5 1046	FLUME-1 GPM 1030	SANDBOIL 1024	FRENCH DRAIN (GPM) 1076	ARTESIAN WELL (EL) 1104	SOUTH RAVINE WEIR (GPM) 1042	REMARKS (Elevations)
02/18/94	Clear	Clear	same	Dry	Clear	231.0	v. active	13.5	1114.6	184.4	None
03/23/94	Clear	Clear	same	Dry	Clear	258.6	Clear	15.6	1113.7	192.5	None
03/29/94	Clear	Clear	15 sec. to fill 1 pint	Dry	Clear	231.0	Clear	3.6	1101.9	161.5	Seepage reduced; Water in higher el. seeps effected most, WL in artesian well below ground level.
04/01/94	Clear	Clear	15 sec. to fill 1 pint	Dry	Clear	231.0	Clear	1.0	1098.5	154.2	
04/08/94	Clear	Almost Dry	Dry	Dry	Clear	231.0	Less Active	0.1	1094.5	127.3	
04/09/94	Upper elevs. w/reduced flow	Almost Dry	Dry	Dry	Clear	231.0	Less Active	0.1	1093.9	127.3	
04/14/94	Same	Almost Dry	Dry	Dry	Clear	191.7	Less Active	0.1	1091.5		
04/16/94	Same	Almost Dry	Dry	Dry	Clear	143.3	Less Active	0.1	1091.0	97.8	
04/21/94	Same	Almost Dry	Dry	Dry	Reduced Flow	143.3	Less Active	0.0	1089.5	84.8	Contractor not pumping waste water, so seepage in area 5 was not flooded and was observed to be barely flowing.
04/28/94	Upper elevs. Dry	Dry	Dry	Dry	Reduced Flow	258.6	Less Active	0.0	1085.0	68.4	Rain, T Storms
05/10/94	Upper elevs. Dry	Dry	Dry	Dry	?	110.4	Less Active	0.0	1082.5	35.5	Area-1 seepage continues to dry out to progressively lower elevations.
05/20/94	Upper elevs. Dry	Dry	Dry	Dry	?	258.6	Reduced	0.0	1076.0	32.6	Cannot see seepage in area 5, flooded with contractors waste water.
06/01/94	Upper elevs. Dry	Dry	Dry	Dry	?	90.1	Reduced	0.0	1075.0	22.5	
06/07/94	Upper elevs. Dry	Dry	Dry	Dry	Dry	46.0	Reduced	0.0	1074.2 (Dry)	17.3	Seepage Area 5 dry, Contractor not pumping, seepage can be observed. Artesian well also dry
06/28/94	Dry	Dry	Dry	Dry	Dry	38.4	Reduced	0.0	1074.2	12.9	Seepage areas dry
07/25/94	Dry	Dry	Dry	Dry	Dry	54.0	Reduced	0.0	1074.2	14.5	
08/18/94	Dry	Dry	Dry	Dry	Dry	18.8	Reduced	0.0	1074.2	2.9	
09/17/94	Dry	Dry	Dry	Dry	Dry	4.7	Reduced	0.0	1074.2	0.0	Completed Cutoff Wall 8/26/94 Seepage has been controlled.

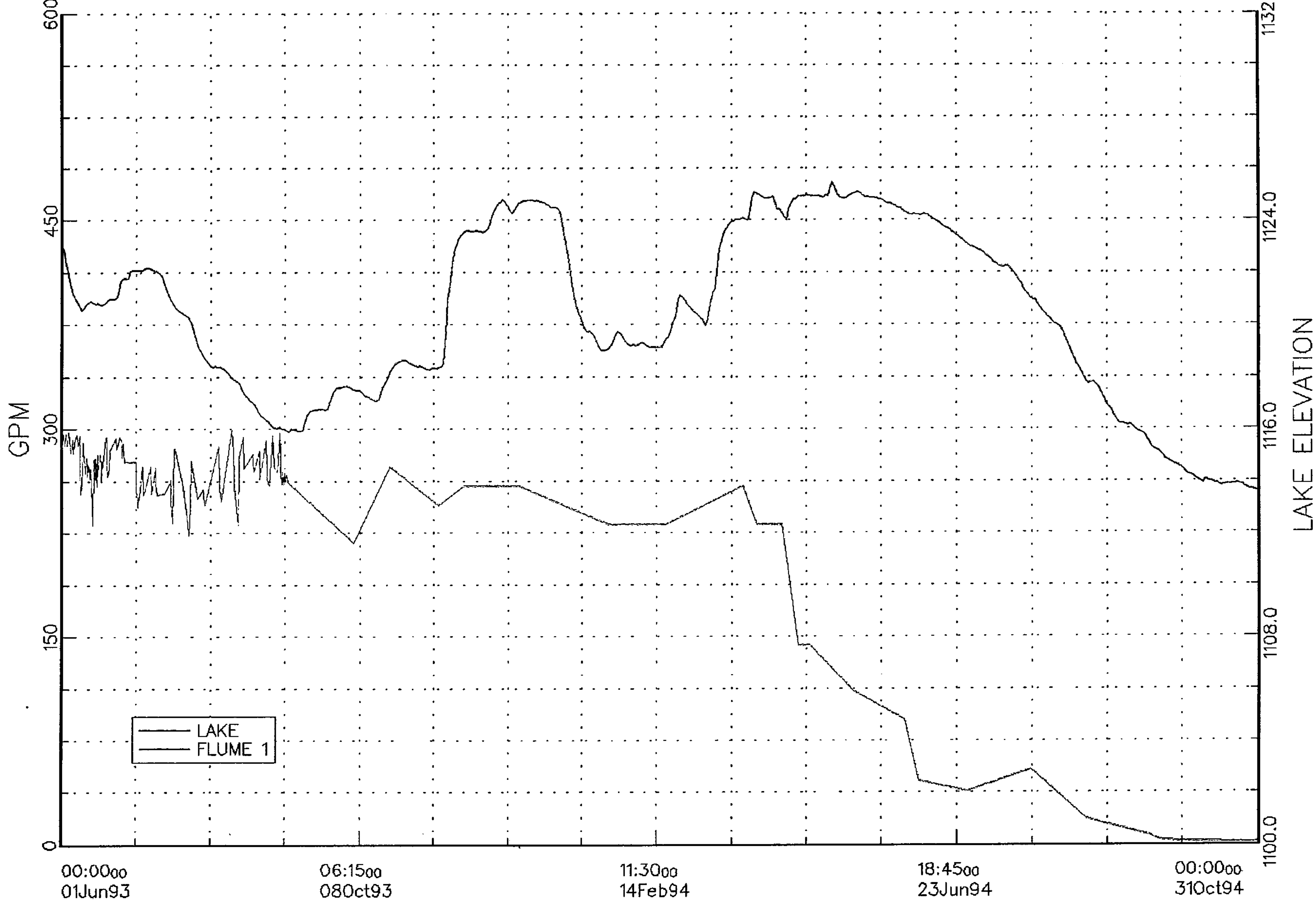

DIKE 1 FRENCH DRAIN FLOW
LAKE ELEVATION
1090
1100
1110
1120
1130
GPM
0.0
5.0
10.0
15.0
20.0
00:00oo 01Sep92
17:00oo 09Mar93
12:00oo 15Sep93
05:00oo 24Mar94
00:00oo 30Sep94
LAKE
FRENCH DRAIN

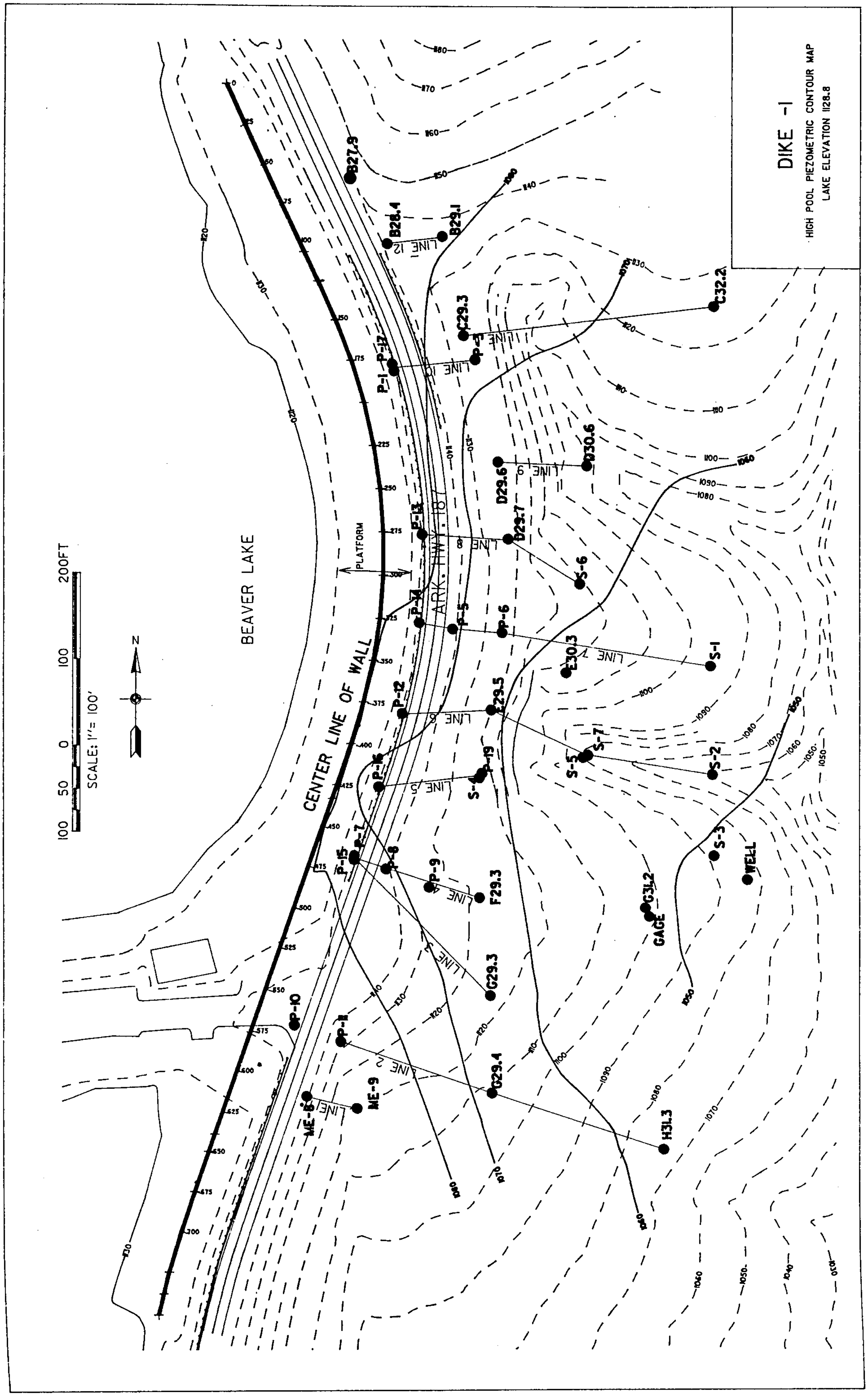


DIKE 1 ARTESIAN PIEZOMETER
ELEVATION
1130
1115
1100
1085
1070
00:00oo 01Jun93
12:00oo 30Sep93
23:00oo 29Jan94
12:00oo 31May94
00:00oo 30Sep94
LAKE
ARTESIAN


DIKE 1 SOUTH RAVINE WEIR FLOW
LAKE ELEVATION
1100.0
1108.0
1116.0
1124.0
1132.0
GPM
0
80
160
240
320
00:00:00
01Jun93
12:00:00
30Sep93
23:00:00
29Jan94
12:00:00
31May94
00:00:00
30Sep94
LAKE
S RAVINE

DIKE 1 FLUME 1





DIKE -1
HIGH POOL PIEZOMETRIC CONTOUR MAP
LAKE ELEVATION 1128.8
BEAVER LAKE
CENTER LINE OF WALL
ARK. HWY. 187
PLATFORM
SCALE: 1"= 100'
100 50 0 100 200FT
N
B27.9
B28.4
B29.1
C29.3
C32.2
D29.6
D29.7
D30.6
E29.5
E30.3
F29.3
G29.3
G29.4
G31.2
H31.3
P-1
P-3
P-5
P-6
P-7
P-8
P-9
P-10
P-11
P-12
P-13
P-14
P-15
P-16
P-17
P-19
S-1
S-2
S-3
S-4
S-5
S-6
S-7
ME-8
ME-9
WELL
GAGE
LINE 1
LINE 2
LINE 3
LINE 4
LINE 5
LINE 6
LINE 7
LINE 8
LINE 9
LINE 10
LINE 11
LINE 12

PLATE C-21

BEAVER DAM COMPLETION REPORT

APPENDIX - D

APPENDIX D: Beaver Cutoff Wall Extension Grout Curtain Completion Report.

Beaver Dam Grout Curtain Construction REPORT

(Constructed 15 May 95 thru 21 July 95)

Grout Curtain Seepage Cutoff Construction Report

1. Introduction.

1.1 Background. Construction of a concrete secant pile cutoff wall at Beaver Dam, Arkansas was completed in August 1994 by Rodio-Nicholson Joint Venture (RNJV). The wall, while successful in cutting off seepage through the weathered and faulted rock under the dike, is located approximately 65 feet upstream of the axis of Dike 1 (Fig-3). At the south end of the wall, this 65 foot offset resulted in the upper 45 to 50 feet of the RNJV wall lying within pervious material of the adjacent main embankment (Fig-3). Since the pervious shell of the main embankment is highly permeable, lake water could flow around the southern terminus of the RNJV wall and potentially resupply water to the foundation of the dam assuming the natural clay overburden material was nonexistent in isolated areas over the weathered Boone Formation rock. Since this could not be proven or disproven, based on thorough review of the Main Embankment contract boring logs and construction drawings, higher authority encouraged the district to install the cutoff wall extension. Based on their concern as well as taking into consideration that the extension will provide additional insurance against a possible end around seepage problem in the future, it was decided to construct the wall extension.

1.2 Location. The grout curtain was constructed at Dam Station 77 + 21.07 (RNJV Pile No. 737). It runs perpendicular from the RNJV wall downstream for 36 feet and ties into the random fill portion of the Beaver Dam Main Embankment at about elevation 1130 (Fig-1).

1.3 Problem. The southern terminus of the impermeable RNJV secant pile wall (completed 1994) ties into pervious materials of the Beaver Dam Main Embankment (Fig-3). This could allow lake water to flow around the southern end of the wall and possibly enter the foundation behind (downstream) the wall.

1.4 Planning Beaver Wall Extension Grout Curtain. A single line grout curtain extending downstream from the RNJV wall and tying into the impervious portion of the dam was the chosen method of blocking seepage around the south end of the wall (Fig-2). The grout curtain was proposed to be constructed by close spaced drilling (grout holes 2 ft. apart), consisting of primary and secondary holes, with further split spacing if necessary and utilizing gravity as the method of grout placement. The pervious shell was believed to be porous enough that gravity grouting could be used to form the grout curtain. Using gravity alone; however, proved to be inadequate, because of the large amount of fines present in some portions of the pervious shell.

It was known from past experience that drilling through the pervious shell portion of the Main Embankment could be a time consuming and expensive task due to both excessive water normally produced and hole instability. Uncertainty existed as to whether grout holes could be drilled through 30 feet of (GP) material

(pervious shell) using water only (mud was not to be used because it would seal the hole preventing grout from entering the formation). It was recognized also that a flexible and somewhat experimental approach would be necessary to complete a grout curtain through the existing materials at the site (Fig-1). As a result, two drilling techniques were planned for the grout curtain holes: Conventional drilling, utilizing both hole casing and downstage grouting as needed to prevent the holes from collapsing or using the ODEX air hammer system that cases the hole as it is drilled. The better of the two techniques, based upon field performance, would be used to complete the grout curtain. Both plans are more completely described below.

1.4.1 Conventional Drilling. Conventional drilling using downstage grouting involved grouting all primary borings in the grout line to a specified depth or elevation and then repeating the process with secondaries, tertiaries, etc. until an entire horizon the length of the grout curtain was consolidated. The borings would then be deepened and grouted to a second stage completing the second horizon. This process would be repeated until the borings were grouted and consolidated to their required depth, completing the final grout curtain. This method, while effective was found to be expensive and time consuming.

1.4.2 Odex Air Drilling. An exemption from ER1110-2-187 from HQUSACE had to be obtained before the ODEX air hammer system could be used in the Beaver Dam Main Embankment. This method was believed faster because the hole was automatically cased as it was drilled, preventing the hole from collapsing. The entire grout hole could then be drilled at one time eliminating the need for downstaging. Once a grout hole was drilled to the full depth intended, the casing would be withdrawn in stages (usually 3 foot increments) with each stage being grouted to refusal if possible. The ODEX drilling technique was intended to be solely an alternative method to conventional drilling; however, it had to be used almost exclusively because of hole instability problems and the associated time and expense of conventional drilling techniques.

1.5 Approval and Funding. The RNJV wall extension was recommended by OCE, approved by the LRD Project Review Board and funded from construction project funds.

2. Mobilization. Obtaining the equipment and personnel to complete the Beaver Grout Curtain was begun 1 May 1995 and extended to 15 May 1995, which was the first production day. Equipment used included property from Little Rock District, Mobile District and the Beaver Resident Office with the exception of the grout plant, which was rented from MMR Enterprises, Inc., Dallas, Texas. In addition, a driller to run the ODEX air hammer was obtained from Mobile District.

3. Equipment Used.

6-ton flat bed truck ....Little Rock District.
12-ton flat bed truck ....Little Rock District.

12-ton flat bed truck	Mobile District.
P/U-1Ton	Little Rock District.
P/U-3/4Ton	Little Rock District.
Dozer-D4	Little Rock District.
Trailer	Little Rock District.
Trailer	Little Rock District.
Trailer	Beaver Res. Office.
Pump, MONO	Little Rock District.
Drill, 314 Failing, Tilt Bed	Mobile District.
Air Compressor, IR900	Mobile District.
ODEX Air Hammer	Mobile District.
End Loader	Beaver Res. Office.
Grout Plant, ChemGrout 550	MMR Enterprises,Inc.

4. Drilling Methods Used and Results. The first hole drilled (P-1) was completed using conventional (roller rock bit and water) drilling techniques. It took a day and a half to complete the drilling phase alone. The second hole drilled (P-6) was completed in approximately 3 hrs of actual drilling time using the Odex Air Hammer system. Air drilling was used almost exclusively from that point on. The rest of the grout curtain holes were ODEX drilled and simultaneously cased through the pervious shell gravel until the casing could be sealed into the upper clay portion of the random fill zone (Fig-1). The ODEX rods and hammer were then removed from the hole leaving the 6-inch casing in place. The bottom of the hole was then drilled conventionally with a 5.75" rock bit, except air was used as the drilling medium instead of water or drilling mud. Large gravel (3" dia) found in the random fill portion of the dam were more effectively ejected from the bore holes using air, which was readily available via the IR 900 air compressor used to run the ODEX hammer. Generally, water could not remove large gravel from the hole. Drilling the grout holes with air instead of water also had an added advantage of allowing water bearing zones and material changes to be detected easier and more accurately. The air extracted cuttings were checked continuously to make geologic logs of the grout holes. The cross section in Figure-1 was derived from these logs. The quantity of cuttings produced by drilling was checked regularly to determine if excessive amounts of fall in or caving was occurring. Caving occurred only one time while constructing the grout curtain, in hole P-4, at which time drilling was stopped and the hole grouted, six days later the hole was redrilled to full depth and grouted without any further problems (Table-1).

The random fill zone was initially believed to consist totally of clays making it impermeable. This was found not true for the location selected for the grout curtain (Fig-1). Half of the random fill consisted of very large gravel, 2-4 inches in diameter. All of the (GP) material and most of the sand initially produced large amounts of lake water. This change of material from what was anticipated resulted in the deepening of several grout holes that were originally intended to be completed at the top of the random fill zone (Fig-2). This can be seen

when the originally predicted depths of holes in Figure-2 are compared to the As-Constructed depths in Figure-1. If the holes had not been deepened, water would have flowed under the grout curtain. It is hoped that the material below the Sylamore Sandstone is relatively impermeable or this could still occur.

Drilling and grouting results for each hole are summarized in Table-1 and a cross section of the grout curtain area is provided in Figure-1. Further information, including drill logs and a fact sheet made on each boring drilled can be obtain from the Soils, Geology and Materials Section at Little Rock District Office.

5. GROUTING.

5.1 Mixes Used & General Procedures. Standard grouting mixes and procedures were used throughout the job. The grout plant mixing tank was calibrated prior to any grout placement to ensure accurate uniformly mixed batches. Grout mixes used on the job are listed below:

3:1
2:1 (WATER : CEMENT RATIO)
1.5:1
1.25:1
1:1
.75:1

The general grouting procedures used for constructing the Beaver Cutoff Wall Grout Curtain Extension are given below:

1. The completed borehole was cleaned of cuttings and debris using air or water.

2. The borehole was set up for pressure grouting (grout header, gages etc. were put in place) and connected to the grout plant.

3. Normally water was pumped into the hole through the grout plant to see how much gage pressure could be developed in the interval being grouted. Knowing an approximate soil permeability helped to determine the starting grout mix.

4. Grout placement was then started.

5. If gage pressure remained constant after a volume of grout was placed, the grout mix was thickened. The process of pumping grout and increasing the grout thickness was continued until the pressure gage began to show a gradual increase in pressure, while grout was being pumped into the hole. If the grouting pressure continued to rise, then the same grout mix was held until the desired grout refusal pressure was reached. After this occurred, the rate of grout placement was slowed down by bypassing a portion of the grout back into placement tank. Gage pressure was maintained by increasing the amount of grout by-

passed until no more grout would go into the hole and grout refusal accomplished. The ODEX casing was then cleaned out and pulled up to the next grouting stop (3-foot increments) where the process was repeated.

If a grout interval could not be grouted to refusal, using the thickest mix possible, then a volume of grout was pumped into the hole (usually 100 cu. ft.) and the maximum gage pressure obtained recorded. Grouting was stopped and the inside of the casing was washed clean of grout. The casing was then pulled up to the next interval (so it would not get stuck) and the placed grout allowed to harden for 1 to 3 hours to seal off this zone. After the grout had hardened, sealing off the lower zone, the upper interval could be pressure grouted. Allowing grout to set up and seal off an interval in the hole that did not reach the desired placement pressure, ensured that most of the grout placed did not go into one weak portion of the hole and that the upper zones could be grouted at higher pressures than the weakest part of the hole.

5.2 Grouting Methods Used and Results. The first two holes (P-1 & P-2) were gravity grouted providing marginal results, with P-1 and P-2 only taking 13 and 31.5 c.f. respectively. Approximately 10 c.f. of grout is theoretically needed to backfill each of these 45 foot deep grout curtain holes, if no caving occurs. Consequently P-1 and P-2 were little more than backfilled with grout. It was calculated that a overall minimum of 1 c.f. of grout per linear foot of hole would be necessary to produce a water proof barrier between holes located 2 feet apart under ideal circumstances. It was also observed in P-2 that a small amount of head pressure (added 3' of casing above the top of the hole) increased the grout take significantly (from 16 to 31.5 cf). A pressure grouting method was clearly needed for an effective grout curtain to be installed. Screwing a casing head into the installed ODEX casing and adding a grout header and pressure gage to the top threads of the casing head produced a good pressure grouting system when joined with the 550 Chem Grout plant. All holes that remained after grouting P-1 and P-2, were grouted under pressures varying from 12 to 150 psi. Normally, the primary holes were grouted to refusal at 25 psi if possible. Secondary holes were grouted to refusal at higher pressures when possible, some exceeded 100 psi.

The material being grouted was soil, usually loose gravel, so there was no real danger of damaging the foundation with excessive pressures. The main problem was controlling the grout, so that most of the grout placed in a hole did not follow one or two paths to the weakest zones of the hole and continue following these weak zones through the foundation material away from the area that was intended to be grouted. This would not only waste cement but also leave the remainder of the hole not grouted or partially grouted. It was found that the best method of determining what was happening during grouting was to watch the pressure gage carefully. If the gage moved somewhat erratically over a period of time (10-15 minutes) building pressure and

dropping again, it was likely that the grouting was effective. The grout being placed was building up pressure, breaking out into new zones and generally saturating the areas that needed grouting. When the placement gage acted in this manner (especially on secondary holes) grout filling was occurring and refusal was close at hand. On the other hand if the grout take was smooth and the pressure remained constant (especially when pumping thick grout mixes) the hole normally could not be grouted to refusal at high pressures (over 25 psi). If a grouting interval could not be brought to refusal at the pressure desired, grouting was stopped after a certain volume had been placed (usually 100 cf).

Using a sanded grout mix would have prevented excessive grout travel and eventually would have blocked off the areas continuing to take grout in a treatment interval. Once the most permeable area was block off with sand, the pressure would rise until the second most permeable zone broke out and began taking grout, and when it became blocked off, the third most permeable zone would begin taking grout until the entire interval being grouted was completed. This solution unfortunately could not be used because the onsite grout plant would not pump sanded grout mixes. The grout plant was also limited in its pumping capacity. Thicker grout mixes could not be pumped fast enough through the 6-inch casing to prevent the grout from setting up causing a blockage. The result was that a 1.25:1 was the thickest mix that could be pumped on hot days (above 90 degrees F) but a thicker 1:1 mix could be used during cooler weather. The grout plant pumping capacity, tremie pipe size and hot weather eliminated the possibility of using thicker mixes to stop the grout from traveling. The only alternative was to use the stop grouting technique, which resulted in a lot of wasted time.

The stop grouting method consisted of grouting a zone and waiting 1-2 hrs for the grout to harden before grouting could resume. The time of the stop varied depending upon the grout mix used to complete the zone, thicker mixes (1:1, 1.25:1) took less than an hour to set up. Usually when a zone was grouted, the casing had be drilled out with a rock bit to prevent the casing from getting grouted in the hole. The casing was then pulled (beat upward) using a 300 pound hammer to the next interval to be grouted, usually up 3 feet. This method worked fine except that more time, effort and cement was probably required. This process of pulling 3 feet of casing, grouting, drilling grout out of the casing, letting the grout harden for and hour and then grouting the next zone was adapted for this job. Consequently most of the time spent completing a hole was during the grouting phase.

6. Statistics of Grout Curtain.

Duration of Project........5 May 1995 to 21 July 1995
Total Horizonal Length.................36 ft.
No. of holes....19 (9 primaries)
(9 secondaries)
(1 tertiary)

Depth.........Varies 28 to 47.4 feet

Liner Feet Drilled............787.0
Grout Placed (cf)..............5403.5 or 6.78 cf/Linear ft.
Bags of Cement Placed.........2800.5 or 3.56 bags/Linear ft
Grout Placed in Pervious Shell (cf)....2495.0 or 46%
Grout Placed in Random Fill (cf).......2908.5 or 54%

7. <u>Effectiveness of Grout Curtain.</u>

Several factors point to the effectiveness of the grout curtain. Holes drilled early on in the grout curtain produced large amounts of water, while some also connected with other holes that had been drilled but not grouted or recently grouted. For example P-2, made a connection with P-6 during drilling and P-7 connected with P-4 that had just been placed. Drilling P-7 resulted in air bubbles rising to the ground surface up to 20 feet away from the borehole. There is no doubt that the area of the grout curtain was highly permeable initially.

The primary holes were drilled first and generally produced abundant amounts of water (Table-2). The last 7 secondary holes drilled (Table-2), produced minimal water during the drilling phase before grouting. Three secondaries that produced more water (S-1,S-6 & S-9) can be explained. S-1 lies between P-1 and P-2 that were not pressure grouted (gravity grouted). Both primaries took small amounts of grout with much of the grout placed filling up the bore holes instead of treating the surrounding soil. S-1 (which was pressure grouted) would probably best be considered as the first primary hole instead of a secondary. S-6 was drilled out of sequence, before any primaries had been grouted on its downstream side, leaving the secondary unconfined. The main embankment slope had not been prepared (notched back or the ramp made) until after the hole was completed, consequently P-7, P-8 and P-9 that were scheduled to be drilled first were inaccessible. S-9 was the last hole of the grout curtain and consequently its downstream side was unconfined.

The last 7 secondaries and the tertiary hole (Table-2) were grouted to refusal at much higher pressures than the previously grouted primaries. These higher pressures were obtainable because the primary holes had already formed a confining wall or barrier. The curtain produced by the primary holes alone was close to impermeable as evidenced by the lack of water produced in the latter secondaries during drilling. The high grouting pressures produced in the secondaries likely filled in any voids left in the primary wall creating a water tight barrier.

The last 7 secondary holes also took less grout normally than the primary holes (Table-2, Fig-4) and would have taken significantly less if the grouting pressures had not been increased. The tertiary hole(TS-2) actually took less grout than indicated because it was necessary to wash the hole out

repeatedly (after each 3' grouting stop) to prevent the casing from becoming stuck (grouted) into the hole.

In addition a piezometer was placed 50' north from the southern terminus of the wall and 14 feet downstream of the RNJV wall center. This piezometer indicated a water level higher than the lake water level illustrating that the water behind the wall had been trapped and could not equalize with the lake level.

8. Problems Encountered.

8.1 Grout Plant Deficiencies. The inability of the on site grout plant to pump a sanded grout mix added to the cost of the project significantly. Without a means to stop the grout from spreading beyond its needed limits extra cement was required and man hours spent waiting for the grout to harden between grouting intervals.

8.2 Grout Plant Pumping Capacities. Either a grout plant with more pumping capacity or tremie pipe smaller than a 6 inch ODEX casing was needed. Thick mixes of grout (thicker than 1:1) set up in the casing before it could be pumped into the formation material.

8.3 Unexpected Material Changes. The Random Fill portion of the Beaver Dam Main Embankment was believed to be clay and impermeable; however, a large portion of it consisted of large highly permeable gravel (GP) that needed to be grouted.

8.4 No Site Preparation. It would have been more expedient if site had been prepared ahead of time.

8.5 Delays Caused by Air Drilling. Air drilling, even with the ODEX system, opens up air channels through permeable material. On some occasions it was necessary to wait for the grout to harden in a hole just placed before another one could be drilled.

9. Lessons Learned.

9.1 Grout Plant Requirements. Unless a grout plant is going to be used for a specific purpose and all variables are known; it is best to obtain a multi-purpose, multi-function, high pumping capacity plant that has both a mixing and placing tank and is capable of pumping sanded grout mixes.

9.2 ODEX System Advantages. It is very doubtful if the job could have been completed without using the ODEX system. In highly permeable and collapsible (GP) type soils, the automatic casing feature is invaluable.

9.3 Importance of Equipment Compatibility. It is important on a grouting job for the equipment to be matched properly. The grout plant on site was very reliable with minimal maintenance

problems; however its pumping capacity was too small for 6" ODEX casing being used as the grout tremie pipe. The result was that thick mixes remained in the casing to long forming blockages.

10. Demobilization. The embankment slope and platform restoration was contracted to Arlie Weems Inc., and was returned its original condition. The equipment that did not belong to Little Rock District Geotech Branch was returned to the owners.

11. Conclusion. The simple grouting job that was begun on 15 May 1995 consisting of standard rotary drilling and gravity grouting evolved quickly into a rather complex operation. Both the ODEX air hammer system and pressure grouting had to be employed in the first week of production. Large amounts of gravel were encountered in the Main Embankment Random Fill that was believed to consist of mainly lean clay. Many of the grout holes that were scheduled to terminate at the top of the random fill had to be deepened. Slightly more grout was actually placed in the random fill (thought to be impermeable) than in the pervious shell portion of the embankment (Fig-3). In addition the 2-foot distance between adjacent boring resulted in major material changes. An example can be seen in the interval between P-3 & P-4 (Fig-1) below the pervious shell zone. Both P-3 and P-4 consisted of predominately gravel, but S-3 located between was composed of clay. With this much radical material change in the relatively short distance of 36 feet (Fig-1), each hole had to be drilled and grout treated on individual basis. It appears that the grout curtain has produced a positive cutoff for seepage around the southern end of the RNJV wall.

CESWL-ED-G
25 Oct 95

Table 1

BEAVER WALL EXTENSION (GROUT CURTAIN)

HOLE NO. P-1 **STATION** 2' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Meth	Depth (FT)	Grt Date	Bags (Cmt)	Grt (Cf)
05/16/95	1131.6	1085	8.5" Aug	0-4	05/17/95	10	13
			6.75" RR	4-47.4			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
N.D.	N.D.	None	0	0	None	Gravity	P-1 was
							concete
							to 41'

* Concrete had spread over to P-1 from the RNJV pile wall for the top 41 feet of the hole. Only the bottom 6' of hole could have taken any grout.

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
N.D.	47.4	1:1	13	10	41-47'	Gravity	Wasted 2
							bags of
							cement.

HOLE NO. S-1 **STATION** 4' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
05/23/95	1132	1086.9	8.5" RR	0-9	05/24/95	232	536
			ODEX	9-37			
			5.75" RR	37-45.1			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	32.5	2:1	320	128	8-32.5'	15-25	Most of
		1.5:1	56	28			Per. shell
		1:1	12	8			grouted to
							25 psi ref.

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
32.5	45.1	2:1	100	40	39-45'	22	Refusal at
		1.5:1	48	26			22 psi.

* See abbreviations at end of Table 1.

HOLE NO. P-2 **STATION** 6' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
05/18/95	1132	1086.4	8" AUG	0-3	05/19/95	14	31.5
			ODEX	3-33			
			5.75" RR	33-45.6			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
9	33	2:1	21.5	8.5	9-33'	Gravity	Wasted 1.5
							bags

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
33	45.6	2:1	10	4	41-45.6'	Gravity	

HOLE NO. S-2 **STATION** 8' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
06/28/95	1132	1086	8" RR	0-6'	06/29/95	4	10
			ODEX	6-33'	07/11/95	105	208
			5.75" RR	33-46			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	32	2:1	95	38	14-17'	60-100+	Refusal at
		1.5:1	19	10	21-32'		all grouting
		1.25:1	104.5	57			stops.

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
32	46	2:1	10	4	None	100	Lower
							zones well
							grouted

*Job shut down 6/29/95 for July 4th until 7/10/95 after lower zone was completed.

*No water produced below 32'.

HOLE NO. P-3 **STATION** 10 D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft.)	Grt Date	Bags (Cmt)	Grt (Cf)
05/22/95	1132	1086.8	7.5" RR	0-3	05/22/95	186	413.5
			ODEX	3-33			
			5.75" RR	33-45.2			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	30.5	2:1	175	70	5-23'	15-25	Most of
		1.5:1	44	22			Perv. Shell
							Grouted to
							Refusal at
							25 psi.

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
30.5	45.2	2:1	100	40	37.5-45.2'	EST.-25	Pressure
		1:1	85.5	54			gage
							broken.

HOLE NO. S-3 **STATION** 12' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
06/21/95	1132	1087	8" RR	0-3	06/21/95	293	546.5
			ODEX	3-39	06/26/95		
			5.75" RR	39-45			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	23	2:1	70	28	8-10'	25-100+	Refusal in
		1.5:1	40	20	13-19'		zone at
		1.25:1	71.5	39			high psi

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
23	45.2	2:1	65	26	23-30	15-20	25 psi not
		1.5:1	32	16	40-45		possible
		1.25:1	115.5	63			
		1:1	135	87			
		.75:1	17.5	14			

HOLE NO. P-4 **STATION** 14' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
05/31/95	1132	1086.5	8" RR	0-3	06/01/95	24	49.5
			ODEX	3-28'	06/07/95	302	625.5
			5.75 RR	28-45.5			
The 24 bags of cement placed on 6/1/95 was to stabalized the hole so it could be completed.							

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	22	2:1	120	48	4-22'	15	25 psi not
		1.5:1	32	16			possible
		1.25:1	33	18			
		1:1	13.5	9			
* Lost 18 feet of casing in hole, results was 0-17' of hole gravity grouted only.							

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
22	45.5	2:1	270	108	28-45.5'	15-17	25 psi not
		1.5:1	64.5	34			possible
		1.25:1	25	15			
		1:1	117	78			
* Large amounts of water produced in lower part of the hole.							

HOLE NO. S-4 **STATION** 16' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
07/15/95	1132	1087	8" RR	0-6	07/15/95	48	120
			ODEX	6-28			
			5.75 RR	28-45			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	23	2:1	35	14	14-23.5'	70	Grouted to
		1.5:1	12	6			Refusal at
		1.25:1	11	6			70 psi
* Small amount of water produced in entire hole, most at 23' above random fill clay							

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
23	45	3:1	35	10	41-45'(?)	50-100+	Grouted to
		2:1	20	8			refusal at
		1.25:1	5.5	3			high press.
		1:1	1.5	1			
* Wet zone at bottom of hole is questionable; material below perv shell essentially dry.							

HOLE NO. P-5

STATION 18' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
05/27/95	1132	1089	8" RR	0-3	05/29/95	54	126
			ODEX	3-26			
			5.75 RR	26-43			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	22	2:1	50	20	2-3'	25	Grouted to
					12-22'		refusal at
							25 psi

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
22	43	2:1	50	20	35-43'	25	Grouted to
		1.5:1	28	14			refusal at
							25 psi

HOLE NO. S-5

STATION 20' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
07/12/95	1133	1088.5	8" RR	0-7	7-13	142	324
			ODEX	7-31			
			5.75 RR	31-44.5			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
9	20	2:1	85	28	10-15'	100	Refusal at
		1.5:1	40	20			100 psi for
		1.25:1	38.5	21			most of
							zone.

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
20	44.5	2:1	95	38	25-33'	100	Refusal at
		1.5:1	16	8	43-44.5'		100 psi for
		1.25:1	49.5	27			most of
							zone.

* No free water produced in this zone. Wet zones are apparently not producing water.

HOLE NO. P-6 **STATION** 22' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
05/18/95	1132	1104	8" RR	0-3	05/20/95	74	182
			ODEX	3-28			
*ODEX used first on this hole, Pressure grouting done first also, no pressure gage.							

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	17	2:1	65	26	6-17	25	Est. psi

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
17	28	2:1	117	48	None	25	Est. psi

HOLE NO. S-6 **STATION** 24' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
05/26/95	1134	1104	8" RR	0-4.5	05/26/95	207	372.5
			ODEX	4.5-21			
			5.75 RR	21-30			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	16	1.25:1	110	60	8-10.5'	15-20	25 psi not
					14-19'		possible
* It is difficult to determine where grout goes that is being pumped, more grout may be in perv.							
shell than indicated.							

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
16	30	2:1	20	8		15-25	25 psi not
		1.5:1	8	4	16-19'		possible
		1.25:1	176	96			usually.
		1:1	58.5	39			
* There was grout taken near BOH. not producing water does not mean no grout takes are poss.							

HOLE NO. P-7 **STATION** 26' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
06/01/95	1134	1088.5	8" RR	0-3	06/01/95	352	685.5
			ODEX	3-33			
			5.75 RR	33-45.5			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	16	2:1	110	44	11-16'	10-15	Most of
		1.5:1	138	72			Perv. Shell
		1.25:1	131	72			grouted to
		1:1	57	38			10-15 psi
		.75:1	27.5	24			refusal

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
16	45.5	2:1	170	74	29-30'	25	Grouted to
		1.5:1	44	22	41-45'		refusal at
		1:1	9	6			25 psi

HOLE NO. S-7 **STATION** 28' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
07/16/95	1135	1102	8" RR	0-6	07/17/95	80	153
			ODEX	6-16			
			5.75 RR	16-33			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	15	1.25:1	55	30	13-15'	15	
*Est 1 gallon of water/min in entire hole.							

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
15	33	2:1	15	6	19-20	15	
		1.5:1	28	14			
		1.25:1	55	30			

HOLE NO. P-8 **STATION** 30' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
06/16/95	1135	1105	8" RR	0-3	06/17/95	237	416.5
			ODEX	3-16			
			5.75 RR	16-30			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
3	13	2:1	75	30	10-13'	12	25 psi
		1.5:1	28	14			refusal not
		1.25:1	66	36			possible
		1:1	72	48			

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
13	30	2:1	15	6	25.5-30'	12	25 psi
		1.5:1	12	6			refusal not
		1:1	148.5	97			possible

HOLE NO. S-8 **STATION** 32' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
07/18/95	1135	1089.5	8" RR	0-6	07/18/95	34	68
			ODEX	6-15	07/18/95	130	235.5
			5.75 RR	15-45.5			
* Encountered mod. seepage (27-32') stopped; Grouted zone on 7/18/95, then continued.							

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
3	13	2:1	30	12	12-13'	30-50	Grout ref
		1.5:1	44	22			at high
		1.25:1	79	48			pressures

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
13	45.5	2:1	20	8	27-32'	100	Grouted to
		1.5:1	8	4	41-45'		refusal at
		1.25:1	134	70			100 psi
* Minimal water produced during redrill (after grouting at 32 feet).							

HOLE NO. P-9 — STATION 34' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
06/13/95	1135	1090.5	8" RR	0-3	06/14/95	146	281
			ODEX	3-22			
			5.75 RR	22-44.5			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
3	8	2:1	50	20	None	5	
		1.5:1	12	6			
		1:1	13.5	9			
* This zone could only be pressure grouted by driving 8" casing approx 6' into the ground							
and applying minimal pressure so it would not leak.							

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	44.5	2:1	60	24	10-12'	18	
		1.5:1	28	14	43-44.5'		
		1.25:1	38.5	21			
		1:1	78	52			
* Lost 9' of casing in hole, as a result zone 9-17 feet not grouted properly.							

HOLE NO. S-9 — STATION 36' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
07/20/94	1135	1088.5	8" RR	0-4	07/20/95	97.5	173.5
			ODEX	4-15			
			5.75 RR	15-46.5			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
3.5	7	2:1	5	2	None	40	
		1.25:1	11	6			

RANDOM FILL + NATURAL MATERIALS BELOW

Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
7	46.5	2:1	30	12	8-10'	20	
		1.5:1	8	4	12-14'		
		1.25:1	119.5	65			
* No water produced during drilling below 14 feet. Only a small amout produced in entire hole.							

HOLE NO. TS−2 **STATION** 9' D/S OF PILE 737

Date Drilled	TOG (EL)	BOH (EL)	Drl. Method	Depth (Ft)	Grt Date	Bags (Cmt)	Grt (Cf)
07/21/95	1132	1100	8" RR	0−6	07/22/95	29	68.5
			ODEX	6−32			

PERVIOUS SHELL

Top (Ft)	Bottom (Ft)	Grt. Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
8	31.5	2:1	55	21	13−16'	150	Refusal at
		1.5:1	13.5	8	19−24'		150 psi
					25−31.5'		at all stops
* Not much water produced totally in hole.							

RANDOM FILL + NATURAL MATERIALS BELOW

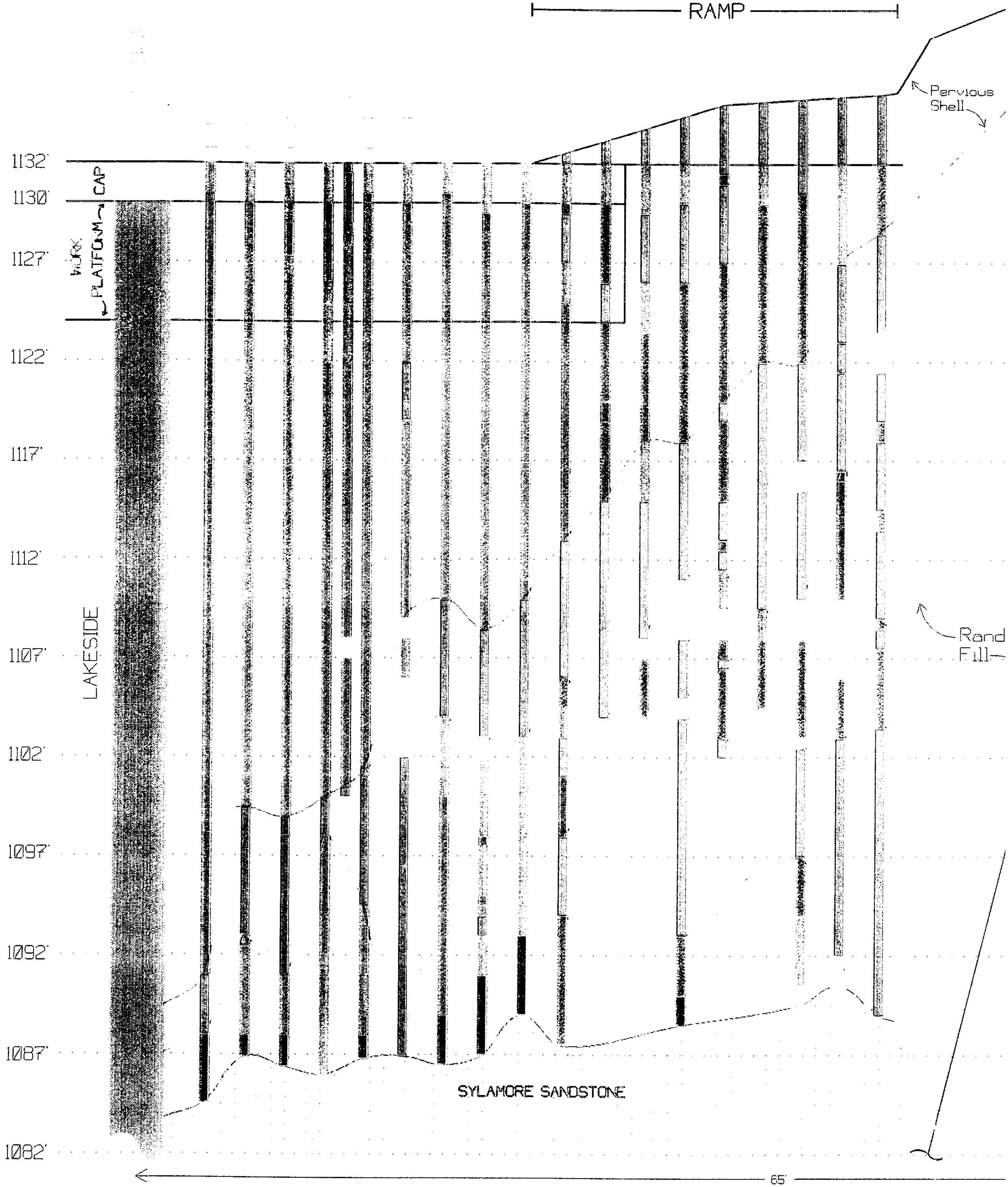
Top (Ft)	Bottom (Ft)	Grt Mix	Grt (Cf)	Bags (Cmt)	Wet Zones	Grt PSI	Remarks
31.5	?		0	0			
* Only drilled half foot into random fill.							
* Est. 1 gal/min. water produced in hole.							

BOH – Bottom of Hole
Drl. Method – Drilling Method
Grt Date – Grouted Date
Bags (Cmt) – Bags of Cement
Grt(Cf) – Grout Placed in Cubic Feet
Grt PSI – Gage Pressure Placed on Grout Entering Top of Hole at Header (pressure produced via pump on grout plant and controlled by grout bypass valve)
Grt Mix – Ratio: Water / Cement of Grout by Volume
RR – Roller Rock Bit

Table 2

SEQUENCE OF HOLES GROUTED

Hole	Date Grouted	Grout(CF)	Grt. Press (PSI)	Water Produced	Remarks
P-1	05/17/95	13	Grav	41-47'	Hole drilled thru solid concrete 0-41'
P-2	05/19/95	31.5	Grav	9'-BOH	Water Produced through out hole.
P-6	05/20/95	182	25	6-17'	Clay from 17'-BOH / Lot of water produced
P-3	05/22/95	413.5	15-25	5-23'/37.5-45.2'	Lot of water produced
S-1	05/24/95	536	15-25	8-32.5'/30-45'	Lot of water produced
S-6	05/22/95	372	15-25	8-10'/14-19'	Moderate amount of water produced.
P-5	05/29/95	126	25	12-22'/35-43'	Lot of water produced
P-4	06/01/95	675	15-17	4-22'/28-45.5'	Lot of water produced.
P-7	06/01/95	685.5	10-25	11-16'/29-30'/41-45'	Moderate amount of water produced.
P-9	06/14/95	281	5-18	10-12'/43-44.5'	Moderate amount of water produced.
P-8	06/17/95	416.5	12	10-13'/25.5-30'	Lot of water produced.
S-3	06/21/95	546.5	20-100	8-10'/13-19'/23-30' 40-45'	Lot of water produced
S-2	06/28/95	218	60-100	14-17'/21-32'	Mod water produced in Perv. Shell, None in rest of hole.
S-5	07/12/95	324	100	10-15'/25-33'/43-44'	Mod water produced in Perv. Shell, None in rest of hole.
S-4	07/15/95	120	50-100+	14-23.5'	Entire hole produced minimal water, est. 1 gal per min.
S-7	07/17/95	153	15	13-15'/19-20'	Entire hole produced minimal water, est. 1 gal per min.
S-8	07/18/95	68	30-100	12-13'/27-32'/41-45'	Entire hole produced minimal water, est. 1 gal per min.
S-9	07/20/95	173.3	20-40	8-10'/12-14'	No water produced below 14', minimal above.
TS-2	07/21/95	68.5	100+	13-16/19-24'/25-31'	Entire hole produced minimal water, est. 1 gal per min.



RAMP
Pervious Shell
1132'
1130'
CAP
WORK PLATFORM
1127'
1122'
1117'
1112'
1107'
LAKESIDE
Rand
Fill
1102'
1097'
1092'
1087'
SYLAMORE SANDSTONE
1082'
65'

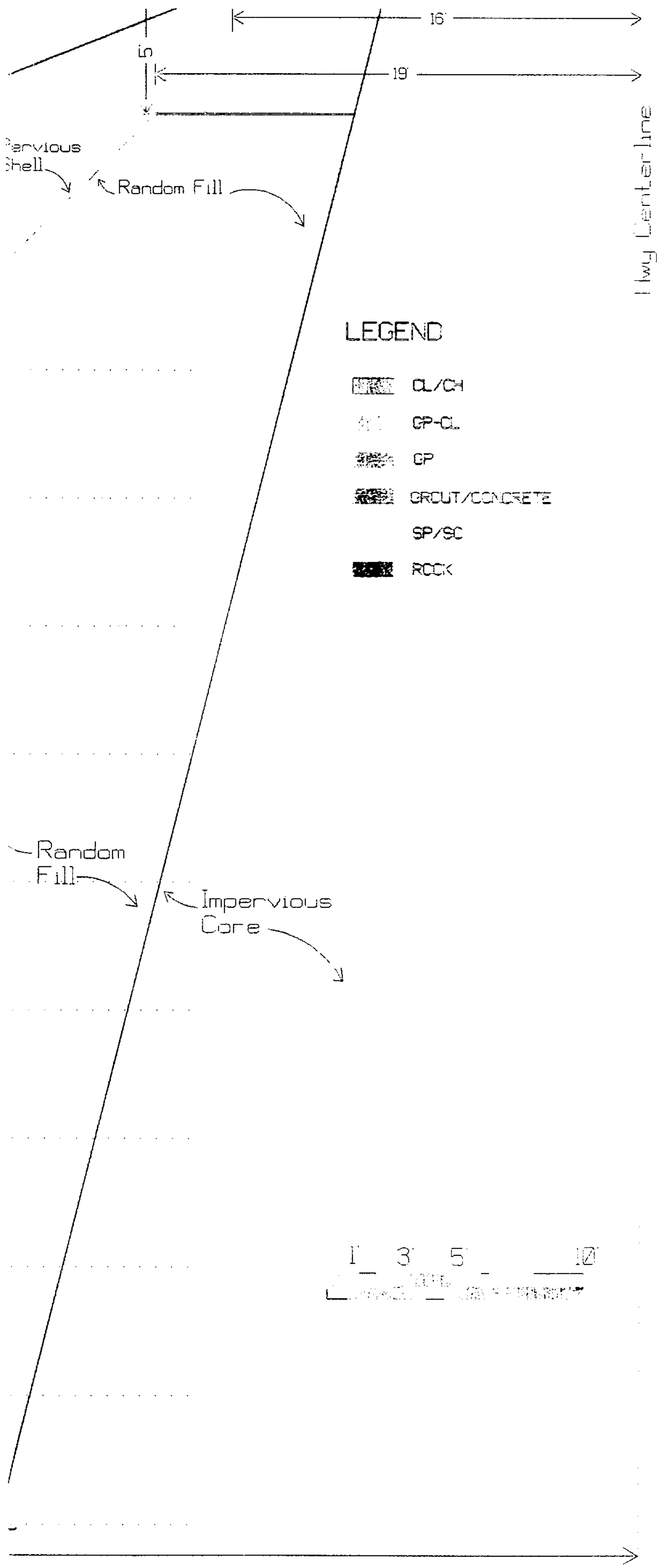
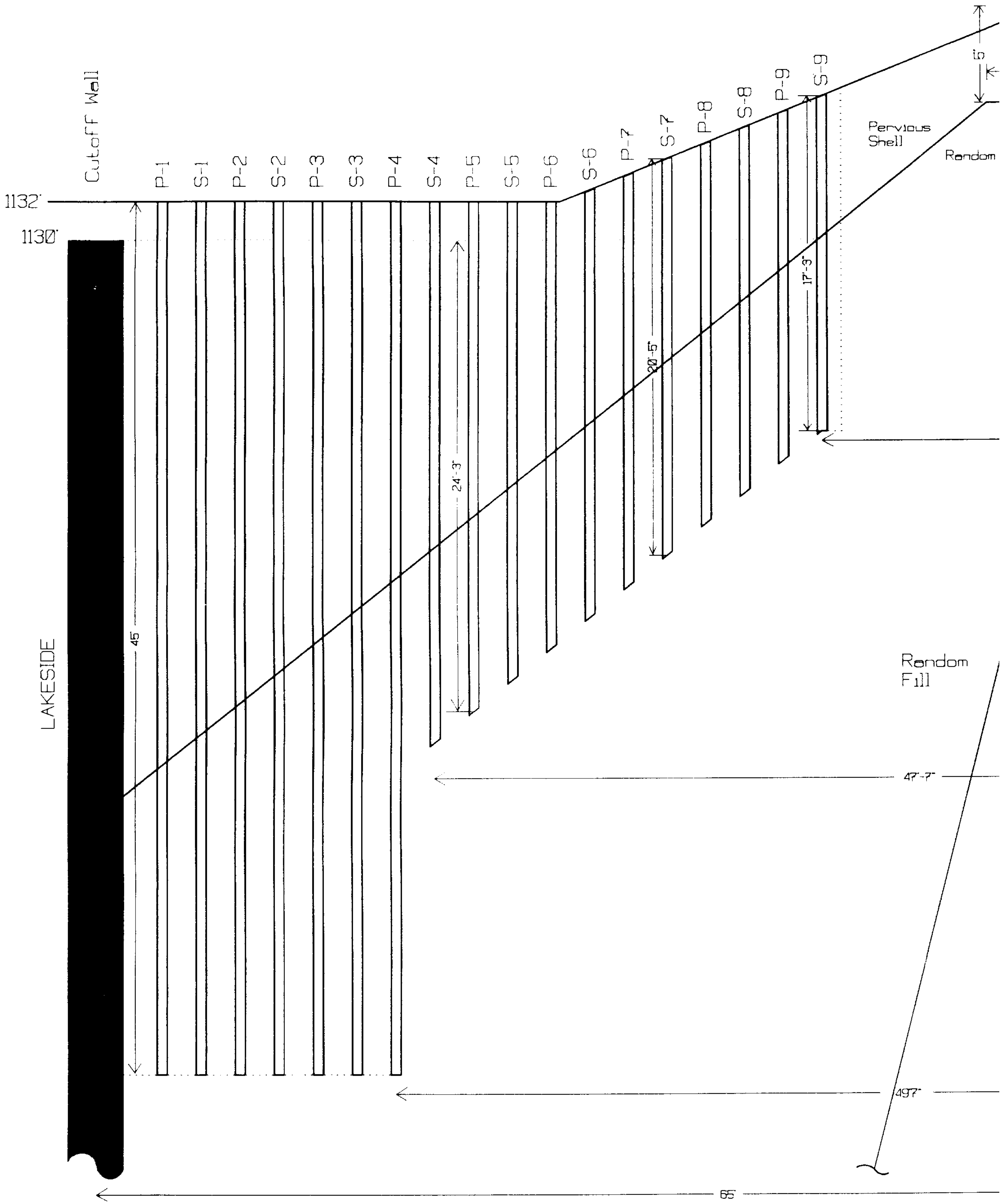


FIGURE 1
CROSS-SECTION
OF AS-BUILT
GROUT CURTAIN


Cutoff Wall
1132'
1130'
P-1
S-1
P-2
S-2
P-3
S-3
P-4
S-4
P-5
S-5
P-6
S-6
P-7
S-7
P-8
S-8
P-9
S-9
5
Pervious Shell
Random
17'-3"
20'-5"
24'-3"
45
LAKESIDE
Random Fill
47'-7"
49'7"
65'

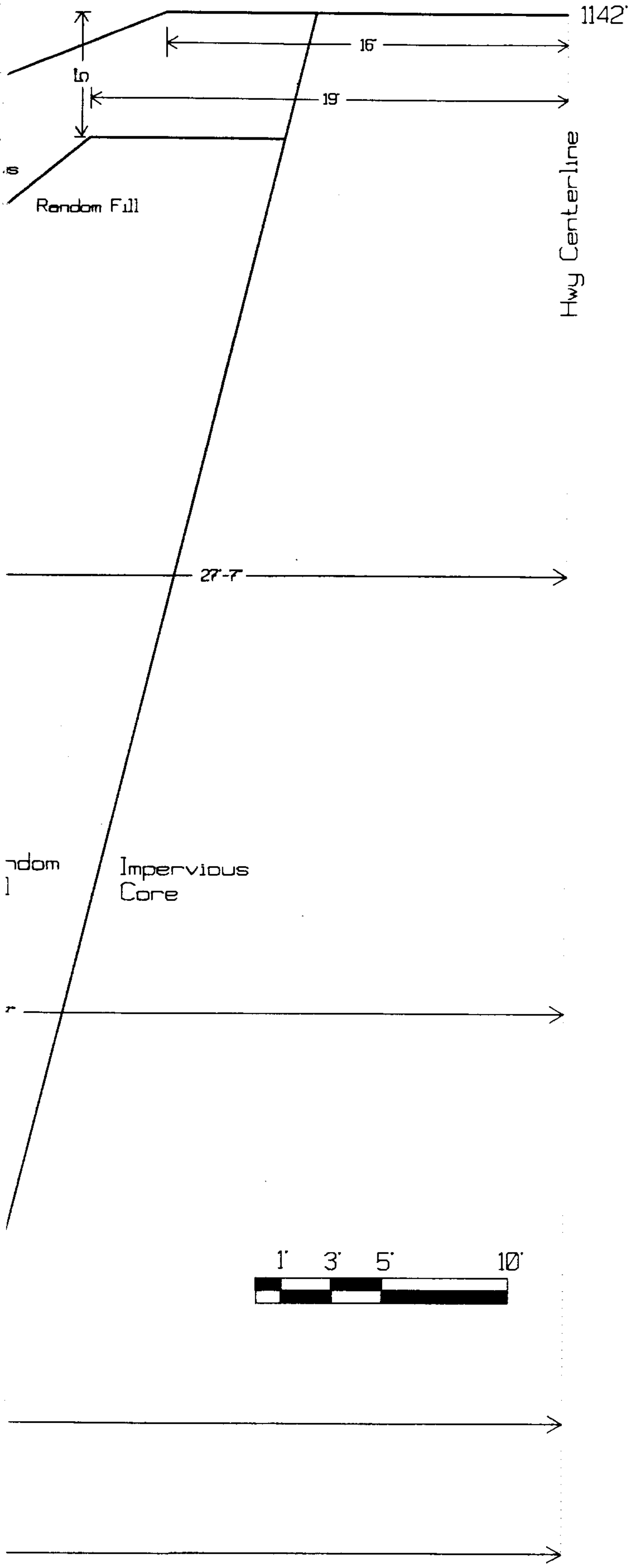
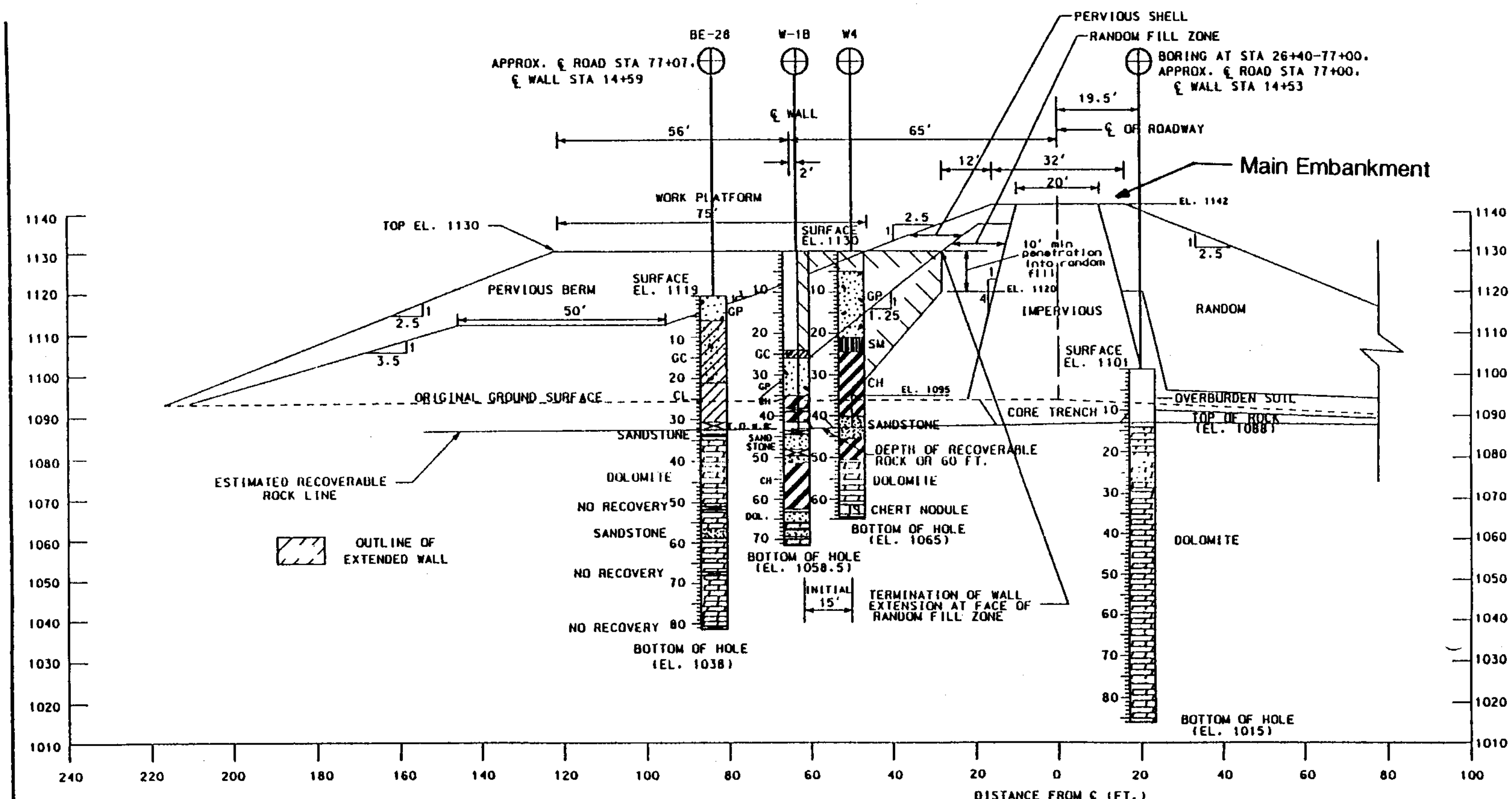


FIGURE 2
ORIGINAL PLAN
OF GROUT CURTAIN

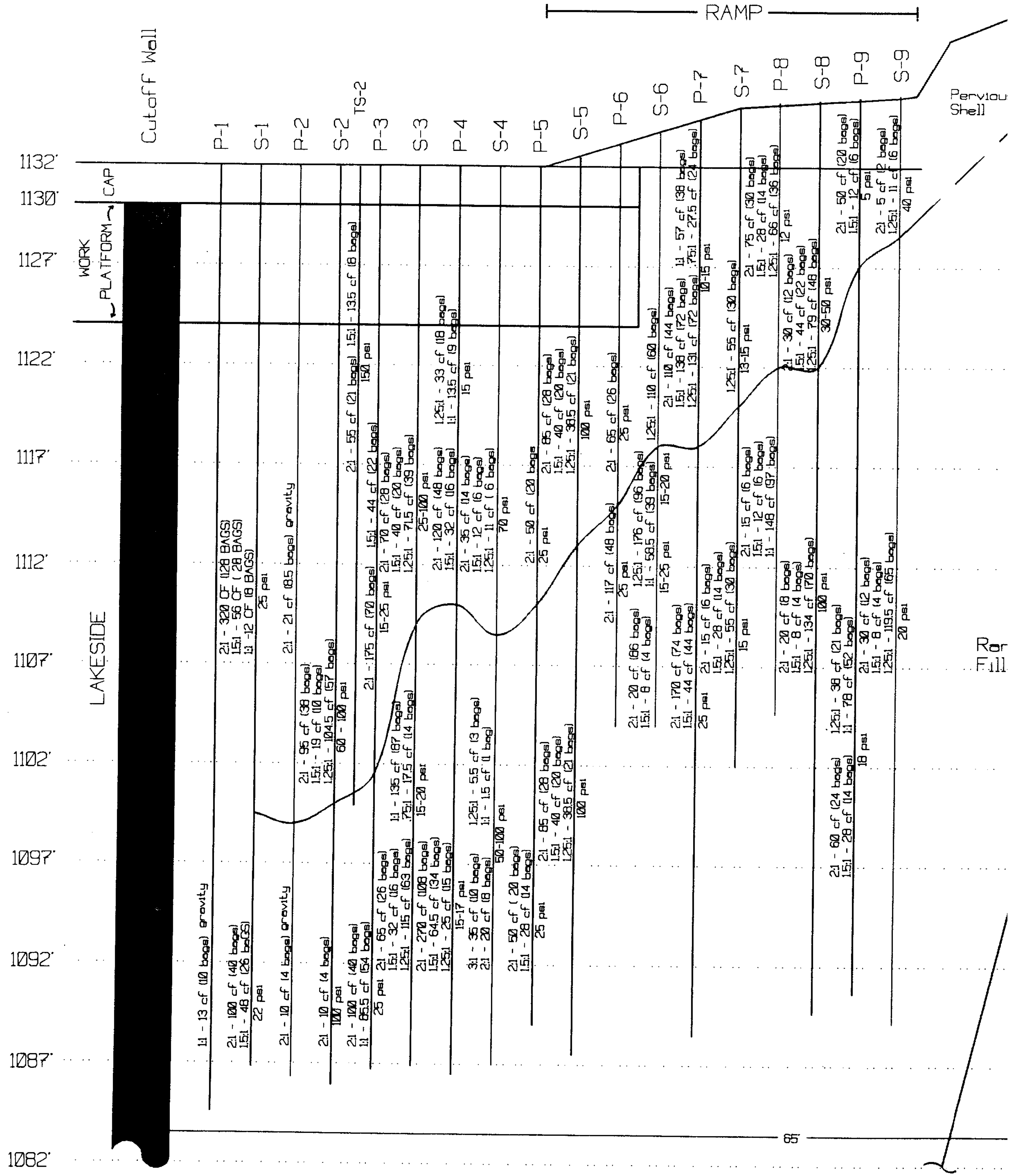


MAIN EMBANKMENT SECTION, (M.E.S.) - GEOLOGICAL PROFILE OF WALL EXTENSION INTO EMBANKMENT, OVERBURDEN, WEATHERED ROCK

(℄ WALL STA 14+75/APPROX. ℄ ROAD STA 77+23)

WORK SAFELY

FIG. 3



RAMP
Cutoff Wall
P-1
S-1
P-2
S-2
TS-2
P-3
S-3
P-4
S-4
P-5
S-5
P-6
S-6
P-7
S-7
P-8
S-8
P-9
S-9
Pervious Shell
1132'
1130'
1127'
1122'
1117'
1112'
1107'
1102'
1097'
1092'
1087'
1082'
CAP
WORK PLATFORM
LAKESIDE
65'

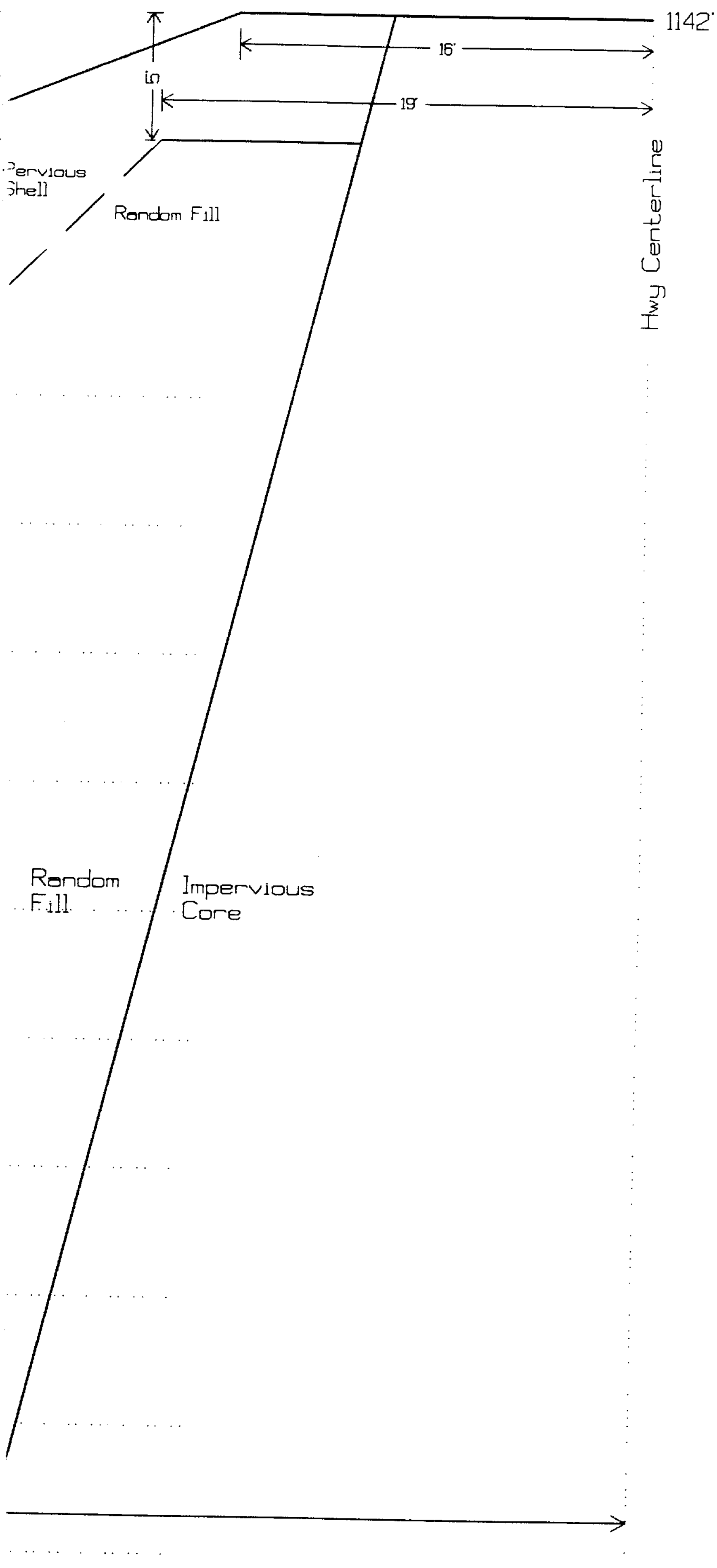


FIGURE 4
GROUT QUANTITY AND
PLACEMENT PSI IN
RANDOM FILL AND
PERVIOUS SHELL

Beaver Dam Grout Curtain Construction Photos

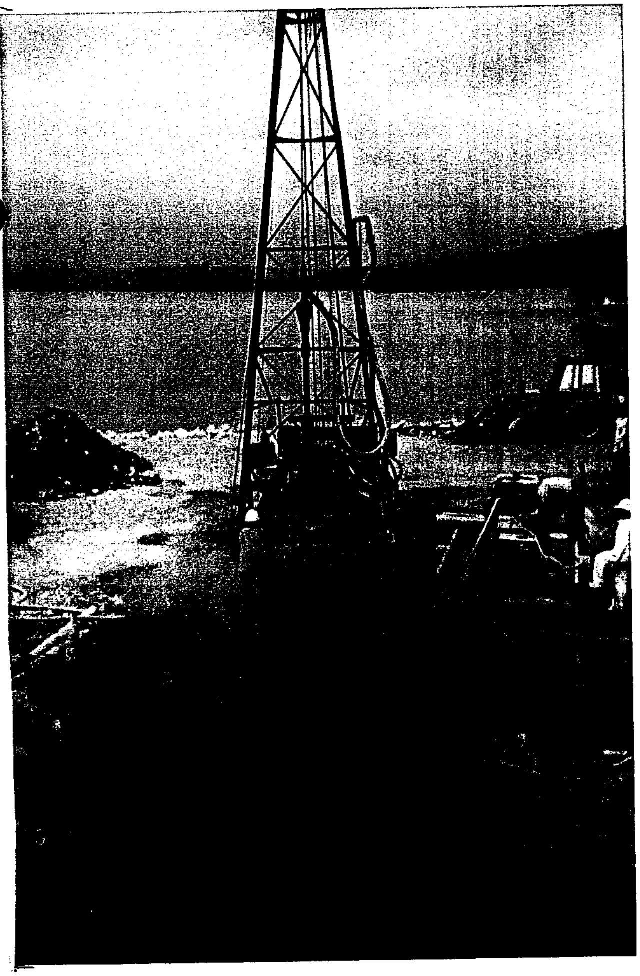


PHOTO NO. 1

Primary Holes like P-5 shown at left produced a lot of water during air drilling.

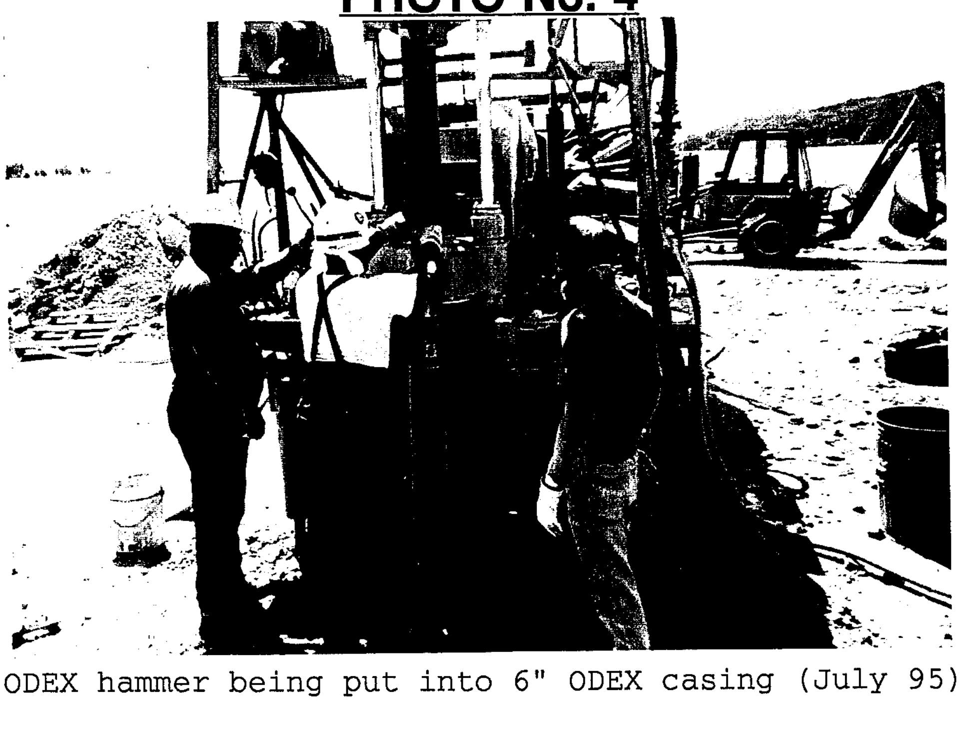
PHOTO NO. 2

Normally, Secoundary Holes like S-4 shown, produced minimal water. (Dust Rises From Hole)

PHOTO No. 3

ODEX air hammer bit with under reamer extended (July 95).

PHOTO No. 4



ODEX hammer being put into 6" ODEX casing (July 95).

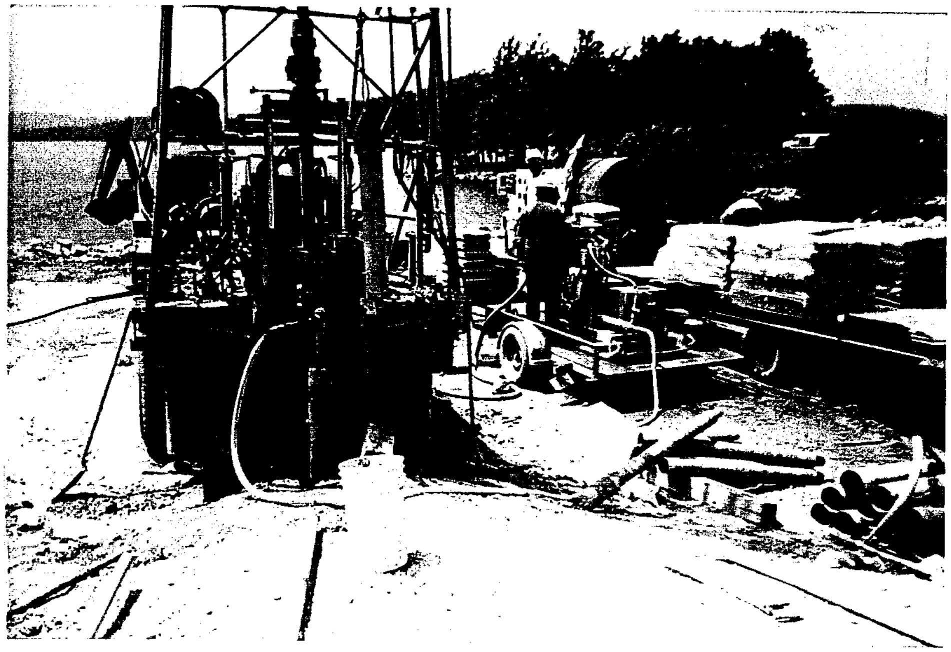
PHOTO No. 5

Large gravel found in the Random Fill Portion of the Beaver Dam Main Embankment.

PHOTO No. 6

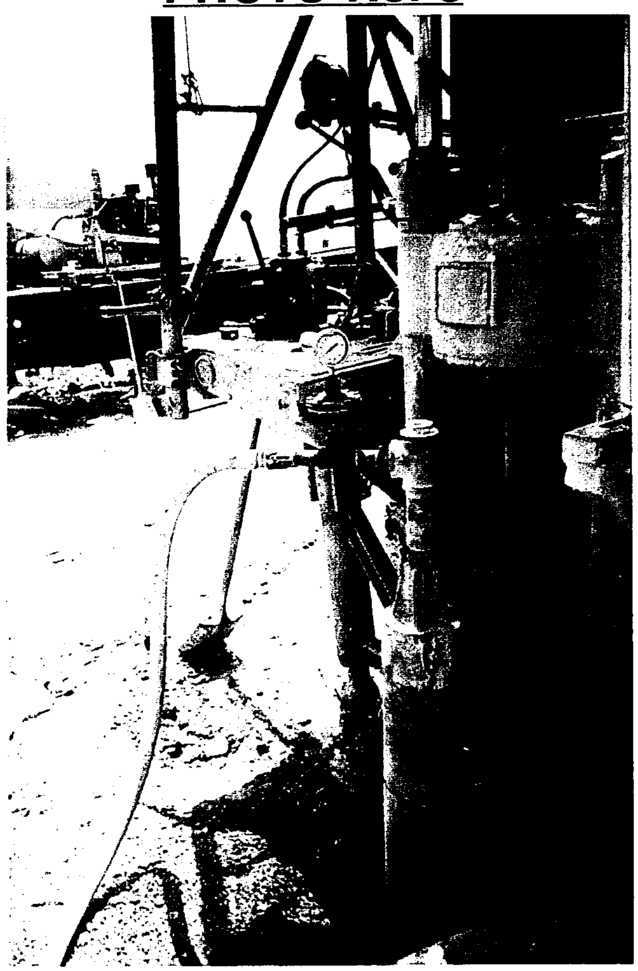
Close up of some of the larger gravel found in Random Fill.

PHOTO No. 7



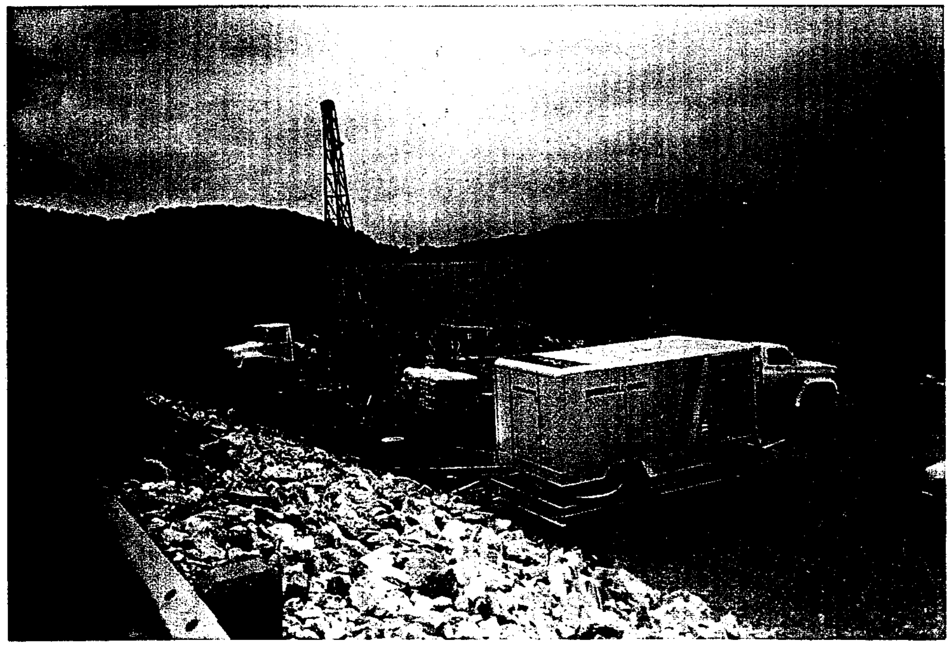
Typical grout placement on Beaver Grout Curtain (June 95).

PHOTO No. 8



Close up of grout header system used on Beaver Dam Grout Curtain (June 95).

PHOTO No. 9



Beaver Grout Curtain site photo (July 95).

PHOTO No. 10

Clay access ramp constructed into Beaver Main embankment (May 95)

BEAVER DAM COMPLETION REPORT

APPENDIX - E

<u>APPENDIX E: PERFORMANCE OF THE CONCRETE CUTOFF WALL DURING HIGH POOLS</u>

CESWL-EG-GS

8 August 1995

MEMORANDUM FOR THE RECORD

SUBJECT: Trip Report Concerning the Performance of the Concrete Cutoff Wall During High Pool

1. <u>Summary</u>. The Lake level at Beaver Dam has remained above 1128' between 9 May and 26 June 1995. Several site investigations made during this period have shown that the seepage cutoff wall remains effective during high lake levels. Current conditions will allow the reservoir water control plan to revert to the original, pre-1986 procedures.

2. <u>Observations</u>.

a. <u>Site Investigations</u>. Beginning on 8 May 1995 and continuing through 26 June, several periods of heavy rainfall created and sustained a lake level above 1128' elevation. Frequent inspections of the historic seepage areas were performed by Geotechnical Branch personnel throughout this period. Natural runoff obscured the actual state of seepage conditions for 2 or more days following a hard rain. Subsequently, all of the historic seepage areas became dry or greatly diminished, obviously independent of the lake level. Only flows through Flume 1, estimated at 10 gpm, remained marginally increased over the rates experienced during lower pools (and periods of normal rainfall). Flume 1 was converted to a V-notch weir to allow measurement of lowered flows resulting from the reduced seepage since completion of the Cutoff Wall. Prior to completion of the Cutoff Wall, flows in excess of 340 gpm would be expected for a pool of 1128'. The recent amounts of heavy runoff have deformed the weir preventing accurate measurements of low flows. Inspection or observation of the slopes of the Main Embankment, Dike 1, and Dike 3 was not possible due to the height of the grass which was in excess of 4' during this period.

b. <u>Piezometer Measurements</u>. Enclosure 1 is a site map of Dike 1 illustrating the location of piezometers in relation to the Cutoff Wall. Piezometer levels dropped dramatically with the completion of the Cutoff Wall. Enclosure 2 lists the chronological trend of several key piezometers with the upper, shaded measurement indicating the start of declining levels. All piezometers monitoring the Dike 1 area now remain a minimum of 34' below lake level. Five piezometers (B29.1B, P-13, ME-8B, ME-9A, and ME-9B) have notably reduced levels but continue to have trends similar to the pool, however, there are indications this may be due to runoff/rainfall infiltration. Future periods with elevated pool levels during dry weather will clarify to which event these piezometers are responding to. Two piezometers, P-6 and P-14, are slowly draining from a falling head test performed several months ago and will require bailing to determine if they remain reactive

to the pool. One of the automated piezometers (P-11) was evidently damaged by fire this past spring during the intentional burn of the Main Embankment grass.

3. Conclusions.

a. Operation of the reservoir under the pre-1986 water control plan is permissible as a result of the effectiveness of the Concrete Cutoff Wall, particularly at elevated pool levels.

b. The Main Embankment, Dike 1 and Dike 3 should be mowed before the grass becomes longer than 12 inches to permit observation and monitoring for seepage (reference CESWL-CO-O memo concerning Mowing of Embankments for Dam Safety, dated 18 November 1994, and ER 1130-2-303 concerning Project Operation Maintenance Guide, dated 15 December 1967). Burning of the grass slopes should be discontinued or be conducted with greater caution to avoid additional damage to automated piezometer equipment.

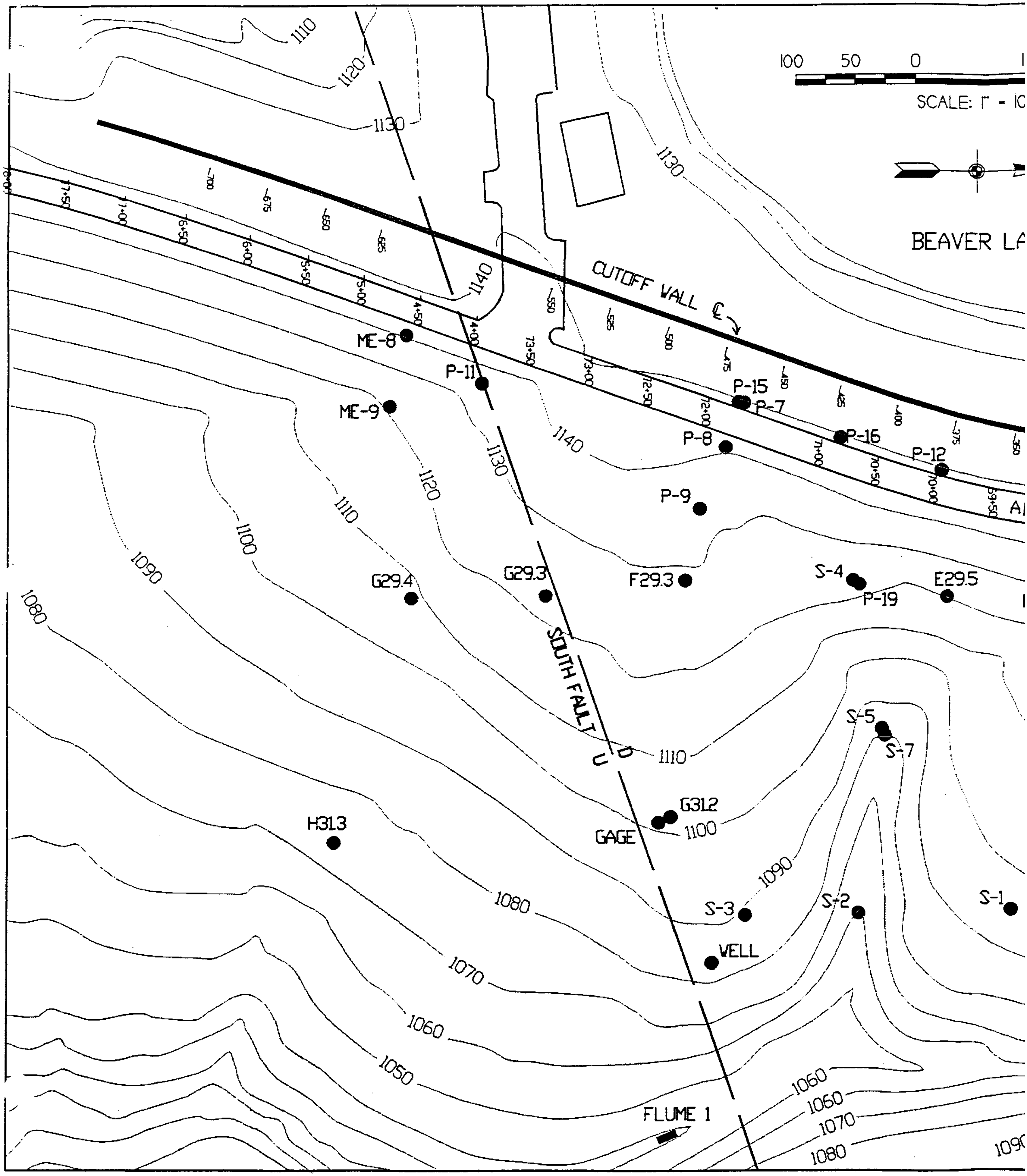
c. Repairs to seepage instrumentation will be completed by Core Drill personnel at the earliest convenience.

Mark Harris

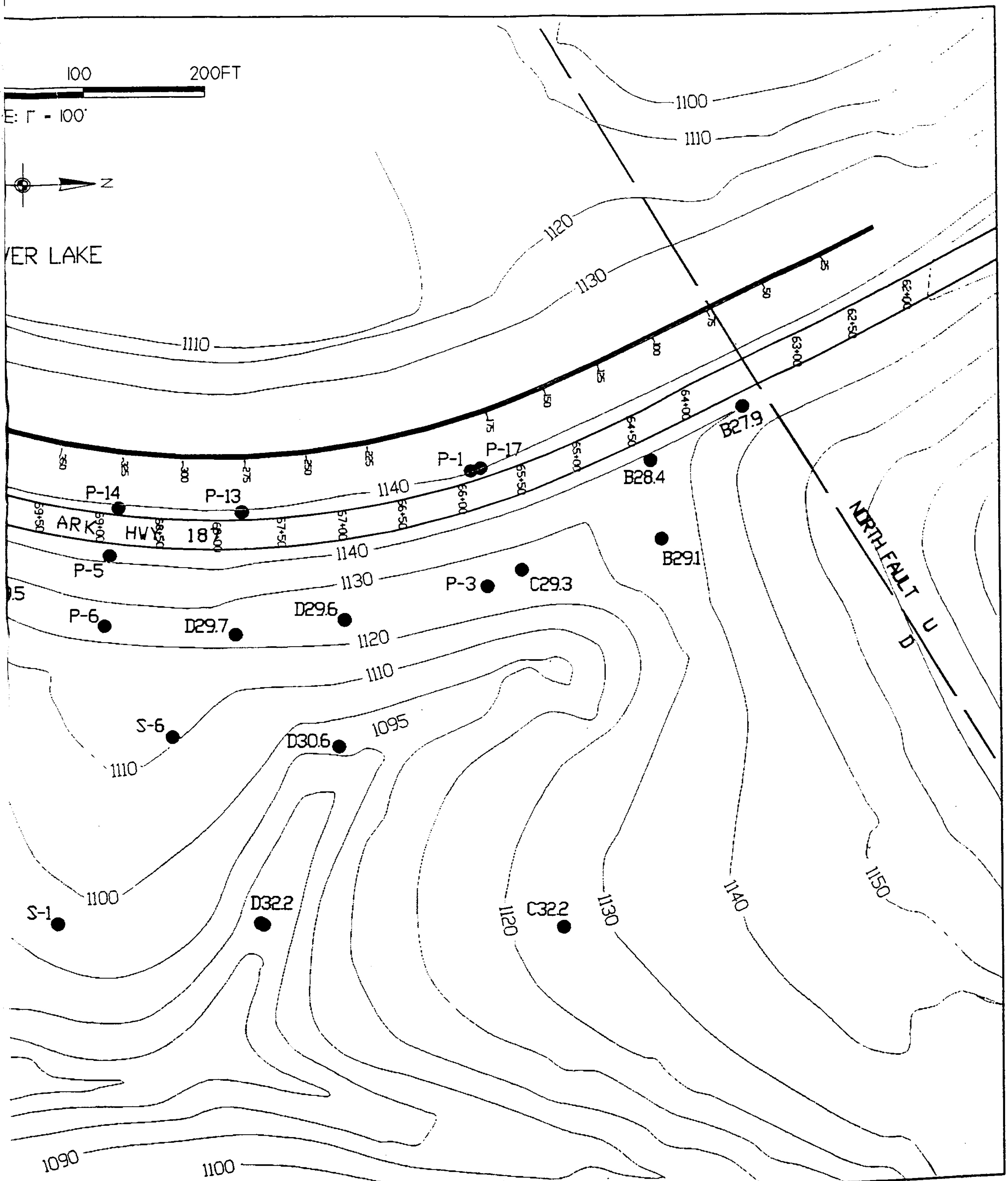
Mark Harris, P.G.
Soils, Geology and Materials Section

Steve Hartung

Steve Hartung, P.G.



100 50 0
SCALE: 1" - 10
BEAVER LA
CUTOFF WALL ℄
SOUTH FAULT
U
D
ME-8
ME-9
P-11
P-15
P-7
P-16
P-12
P-8
P-9
F29.3
S-4
P-19
E29.5
G29.4
G29.3
S-5
S-7
G31.2
GAGE
H31.3
S-3
S-2
S-1
WELL
FLUME 1
1050
1060
1070
1080
1090
1100
1110
1120
1130
1140



100
200FT
E: 1" = 100'
Z
VER LAKE
ARK HWY 18
NORTH FAULT
U
D
1100
1110
1120
1130
1140
1150
1095
1090
P-1
P-17
P-14
P-13
P-5
P-6
P-3
B27.9
B28.4
B29.1
C29.3
D29.6
D29.7
S-6
D30.6
S-1
D32.2
C32.2

ENCLOSURE 1

DATE	Lake El.	Fr Drain	VNotch	WELL	E29.5B	F29.3B	G29.3B	G29.4B	G31.2B	ME-8B	ME-9B	P-11	P-14	P-15	P-16A	P-19	P-9	S-2	S-5	DRY PIEZOS	TIP ELEV.
					******************					******PIEZOMETER ELEVATION******									******************		
20-Mar-94	1124.0			1049.0	1079.2	1113.6	1113.3	1079.8		1109.9	1109.9	1115.6	1099.2			1099.2	1113.3		1069.7		
21-Mar-94	1124.0			1049.1	1079.2	1113.4	1113.0	1079.7		1110.1	1110.0	1115.7	1099.4			1099.1	1113.0		1069.8		
22-Mar-94	1124.0			1049.0	1079.1	1112.9	1112.2	1079.5		1110.0	1109.5	1115.4	1099.5			1099.0	1112.6		1069.8		
23-Mar-94	1124.1			1049.4	1078.8	1111.2	1111.1	1079.6		1109.3	1108.9	1115.5	1099.3			1098.4	1110.7		1069.6		
24-Mar-94	1124.0			1048.8	1078.5	1110.3	1109.8	1079.4		1108.3	1108.6	1114.4	1099.4		1110.4	1098.1	1109.8		1069.4		
28-Mar-94	1125.1	gpm		1048.8	1077.7	1107.4	1107.0	1079.0		1106.1	1107.2	1112.2	1099.4	1114.1	1107.9	1095.6	1106.9		1069.0		
29-Mar-94	1125.0	20.0	110.0	1048.6	1077.4	1106.3	1106.2	1078.6		1105.2	1107.0	1111.6	1099.4		1106.7	1095.2	1105.5		1068.9		
31-Mar-94	1124.8			1048.1	1076.7	1102.5	1102.1	1078.2		1103.7	1105.0	1109.9	1099.4			1094.0	1102.0		1068.4		
02-Apr-94	1124.8			1047.8	1075.7	1099.1	1098.8	1077.7	1065.9	1099.9	1103.4	1106.8	1099.2		1104.0	1091.8	1098.6		1067.7		
10-Apr-94	1124.2		84.0	1046.9	1073.1	1092.3	1092.1	1076.5		1095.2	1100.5	1099.3	1098.5			1085.9	1091.8		1066.0		
15-Apr-94	1124.8		67.0	1047.0	1072.1	1089.4	1089.2	1076.0		1093.8	1099.6	1096.6	1098.2			1083.5	1088.9		1065.5		
19-Apr-94	1124.9	no flow	58.0	1046.5	1071.4	1087.8	1087.6	1075.3	1062.2	1092.9	1098.7	1095.0	1097.8			1082.2	1087.3		1064.6		
23-Apr-94	1124.9		50.0	1046.4	1070.0	1085.9	1086.0	1074.6	1061.7	1092.5	1097.9	1093.5	1097.4	1110.8		1080.8	1085.8	1057.1	1063.9	B27.9A	1095.0
28-Apr-94	1124.9			1044.3	1068.7	1083.8	1083.9	1073.9		1089.0	1095.2	1091.4	1096.7	1110.1		1079.1	1079.0	1056.9	1063.0	B29.1A	1105.9
09-May-94	1124.9		30.1	1045.8	1066.8	1080.3	1080.5	1073.0	1059.6	1086.4	1093.5	1087.5	1095.5	1108.3		1076.3	1080.2	1056.3	1061.8	C29.3A	1096.0
16-May-94	1124.9		20.0	1045.7	1066.0	1078.5	1078.7	1072.5		1085.9	1092.7	1085.3	1094.8	1107.1		1075.1	1078.4	1056.1	1061.3	C32.2A	1087.5
25-May-94	1124.6		17.0	1045.5	1065.1	1076.4	1076.6	1071.6		1084.9	1091.4	1082.8	1093.7	1105.4		1073.7	1076.3	1055.6	1060.6	D29.6A	1094.0
10-Jun-94	1124.2			1045.6	1064.3	1072.7	1072.9	1070.3	1057.3	1081.5	1088.6	1079.3	1092.1	1102.5	1103.6	1071.8	1072.7	1056.1	1060.5	D29.7A	1084.4
20-Jun-94	1123.7		11.5	1045.3	1063.4	1070.7	1070.9	1069.6		1080.6	1088.1	1077.2	1091.1	1100.6	1103.6	1070.7	1070.7	1054.9	1059.5	D29.7B	1064.4
30-Jun-94	1122.9		8.0	1045.2	1062.8	1068.8	1069.1	1069.2	1056.0	1079.7	1086.4	1075.3	1090.1	1098.7	1103.6	1069.6	1068.8	1054.8	1072.8	E29.5A	1086.5
12-Jul-94	1122.2		5.5	1045.2	1062.3	1067.0	1067.3	1068.5	1055.5	1079.1	1086.0	1073.4	1089.1	1096.5	1103.7			1054.3	1072.2	F29.3A	1102.0
27-Jul-94	1120.9			1045.3	1062.0	1065.3	1065.6	1068.1	1055.1	1078.9	1087.9	1071.8	1087.7	1093.9	1103.6			1055.7	1073.1	G29.3A	1105.8
11-Aug-94	1119.1		9.0	1044.1	1061.5	1064.1	1064.4	1067.6	1054.6	1078.8	1086.8	1070.8	1086.6	1091.5	1103.7	1065.9	1063.9	1054.9	1072.0	G31.2A	1073.7
17-Aug-94	1118.0		2.2	1043.9	1061.1	1063.4	1063.8	1067.0		1078.5	1084.7	1070.2	1086.1	1090.5	1103.6	1065.4		1053.6	1071.4	GAGE	1074.1
05-Sep-94	1116.1	0.0		1035.6	1060.8	1061.8	1062.1	1080.3	1053.7	1077.7	1083.4	high from	1084.7	1087.8	1103.7	1061.6		1053.3	1055.2	P-12B	1088.7
26-Sep-94	1114.6	0.0	0.0	erratic	1060.2	1060.1	1060.3	erratic	1052.9	1073.5	1082.3	piez test	erratic	erratic	1103.7	1060.5		1052.5	1054.2	P-16A	1103.3
17-Oct-94	1113.9	0.0	0.0	983.7	1060.0	1059.2	1059.6	1065.3	1052.6	1074.4	1082.9	1082.7	1093.9	1114.9	1103.6	1059.9		1052.4	1053.4	P-1	1103.3
31-Oct-94	1113.6	0.0	0.0	erratic	1059.6	1058.5	1058.8	1064.5	1052.1	1073.7	1082.2	1078.7	1092.1	1109.3	1103.6	1059.4		1052.0	1051.9	P-3	1096.3
16-Nov-94	1120.2			883.0	1059.9	1059.0	1058.8	1066.3	1052.7	1084.7	1087.9	1075.9	1090.3	1101.2	1103.7		1058.5	1052.9	1054.7	P-5	1085.5
30-Nov-94	1119.6			883.0	1060.0	1059.2	1058.9	1066.4	1052.7	1084.3	1087.0	1074.0	1088.9	1096.7	1103.8			1052.5	1053.9	P-7	1085.4
07-Dec-94	1119.7			883.0	1059.8	1058.9	1058.6	1066.4	1052.4	1083.8	1086.3	1073.1	1088.0	1094.6	1104.0			1052.2	1053.2	P-8	1084.5
05-Jan-95	1119.6			883.0	1060.0	1059.0	1058.8	1066.2	1052.6	1083.1	1085.5	1071.1	1085.9	1088.1	1103.9			1052.4	1053.1	S-4	1077.3
24-Jan-95	1123.0				1060.3	1059.4	1059.2	1067.1	1052.9	1086.1	1089.4	1070.2	1084.5	1084.5	1103.7			1053.0	1055.0	S-6A	1073.4
30-Jan-95	1121.3				1060.4	1059.7	1059.4	1067.1	1052.9	1085.5	1088.5	1070.0	1084.1	1083.5	1103.7			1052.8	1054.9	S-7	1053.9
06-Feb-95	1120.9				1060.4	1059.7	1059.4	1066.8	1052.8	1084.7	1087.3	1069.7	1083.7	1082.4	1103.7			1052.6	1054.5		
01-Mar-95	1119.6				1060.2	1059.3	1059.0	1067.7	1052.8	1083.5	1086.3	1069.2	1082.7	1079.4	1103.9			1052.8	1054.7		
31-Mar-95	1119.2			1043.3	1059.7	1058.3	1058.5	1068.1	1051.2	1083.6	1085.4	DAMAGED	1082.4	1076.6	1065.7	1059.5	1058.1	1052.5	1054.4		
01-May-95	1122.4			1044.4	1059.9	1058.7	1058.9	1068.1	1051.5	1085.5	1087.7		1081.4	1075.7	1065.6	1059.8	1058.5	1053.0	1054.9		
11-May-95	1128.5			1044.4	1060.3	1059.8	1059.9	1070.4	1051.8	1088.0	1089.8		1081.4	1075.8	1066.6	1060.5	1059.6	1053.1	1055.1		
29-May-95	1128.6			1044.4	1060.5	1061.1	1061.2	1068.9	1052.2	1090.8	1090.6		1081.4	1076.0	1068.7	1061.3	1060.8	1053.0	1054.9		
06-Jun-95	1129.0			1044.2	1060.2	1061.0	1061.1	1068.3	1052.0	1090.6	1090.3		1081.1	1075.8	1068.4	1061.1	1060.8	1052.8	1054.7		
13-Jun-95	1129.1			1044.8	1060.5	1061.8	1061.8	1069.2	1052.5	1092.1	1092.9		1081.2	1075.9	1068.8	1061.6	1061.6	1053.2	1055.1		
19-Jun-95	1128.8			1144.7	1060.7	1062.4	1062.2	1068.6	1052.6	1091.6	1091.4		1081.3	1076.0	1069.1	1061.9	1061.9	1053.1	1055.0		
26-Jun-95	1128.3			1044.2	1060.4		1062.0	1068.3	1052.3	1090.1	1089.9		1081.0	1075.8	1069.0	1061.7	1061.8	1052.6	1054.4		
12-Jul-95	1126.6			1042.3	1060.1	1064.1	1061.3	1067.6	1052.0	1086.8	1086.8		1081.0	1075.9	1068.3	1061.2	1060.9	1052.5	1054.3		
20-Jul-95	1126.1			1043.9	1059.9	1060.4	1060.7	1067.2	1051.8	1082.9	1085.6		1080.9	1075.8	1067.8	1060.8	1060.4	1052.3	1054.1		
24-Jul-95	1125.8			1038.2	1059.8	1060.4	1060.4	1066.9	1051.7	1079.4	1085.3	1069.4	1080.8	1075.8	1067.6	1060.6	1054.1	1052.2	1053.9		
08-Aug-95	1123.4	0.0	0.0	1038.2	1059.5	1059.3	1059.4	1066.1	1051.3	1083.9	1084.2		1080.6	1075.9	1066.8	1059.9	1059.1	1052.0	1053.0		
Net Change		-20.0	-110.0	-11.2	-19.7	-54.3	-53.9	-13.7	-14.6	-26.1	-25.8	-46.1	-18.8	-38.2	-43.6	-39.1	-54.2	-5.1	-16.8		
Tip El		GPM	GPM	883.0	1046.6	1047.4	1049.1	1041.4	1039.6	1073.1	1071.4		1022.5	1041.3	1056.0	995.2	1042.8	1031.2	1013.1		

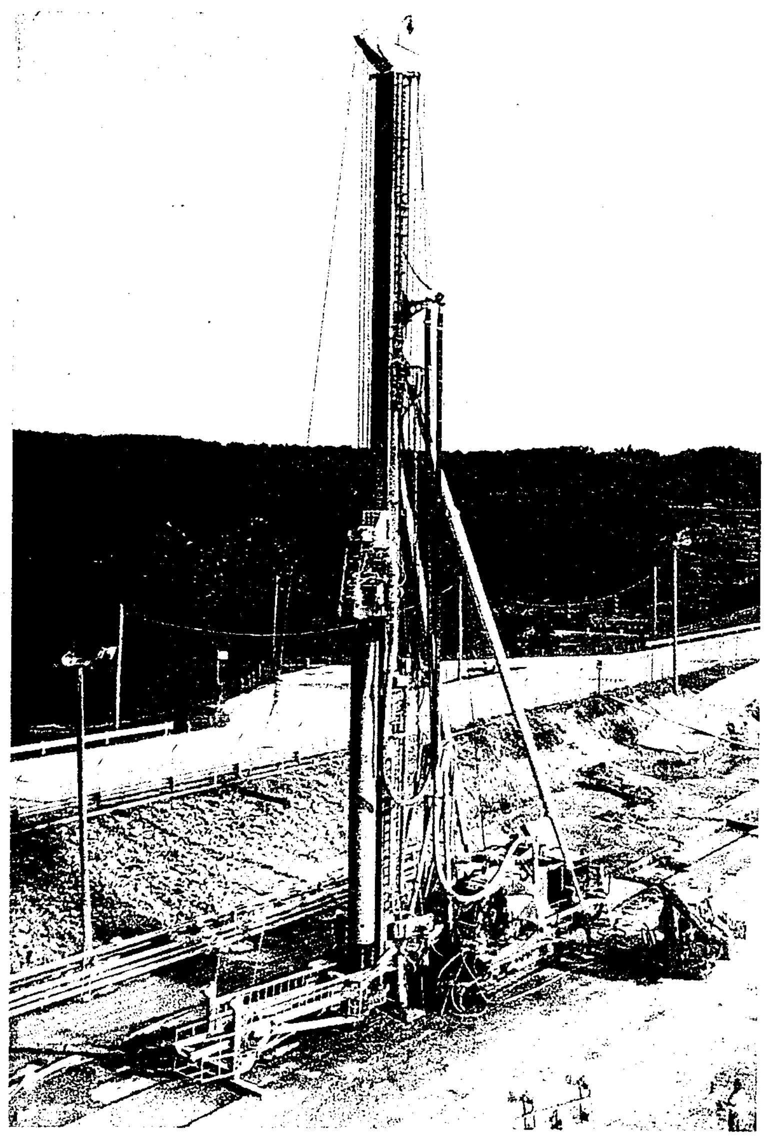
ENCL 2

BEAVER DAM COMPLETION REPORT

APPENDIX - F

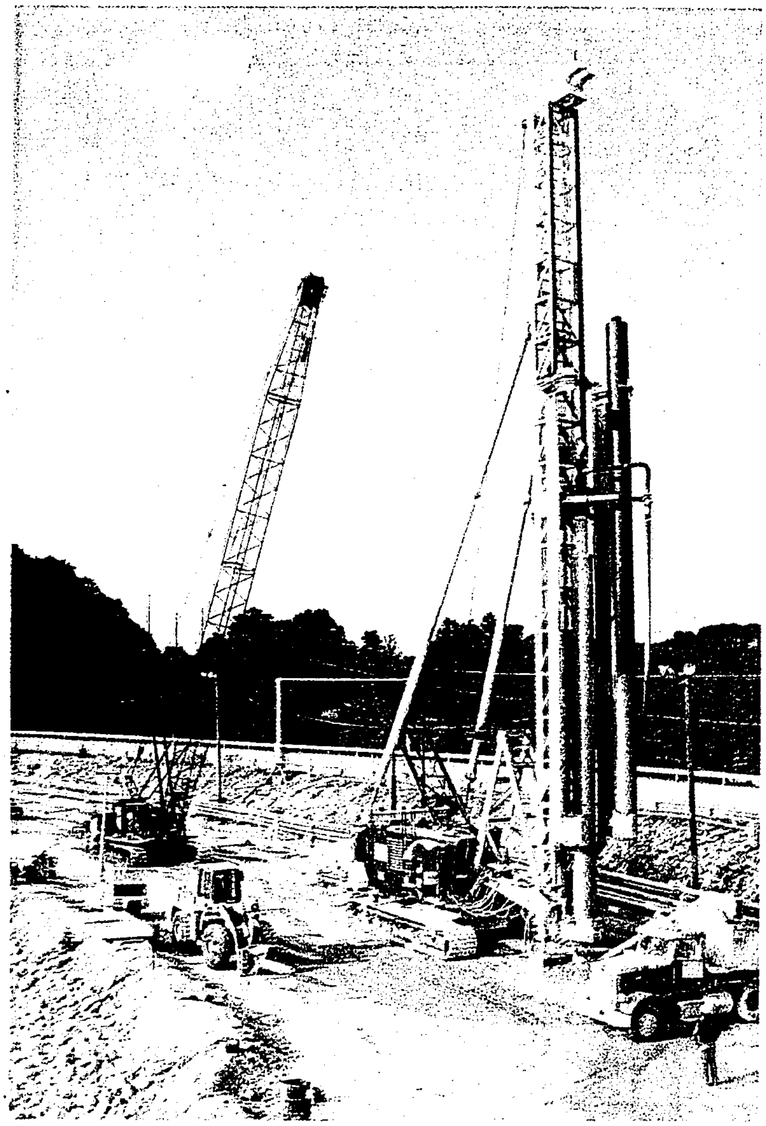
Appendix F: CONSTRUCTION PHOTOGRAPHS

PHOTO No. 2



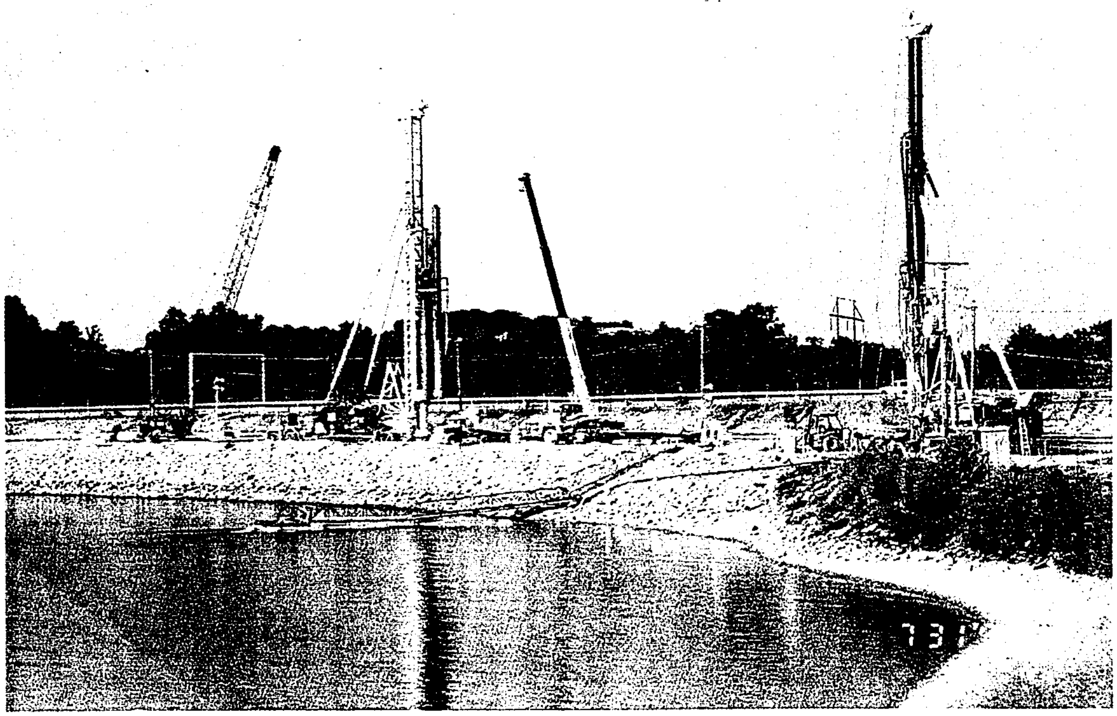
Drilling by drill rig No. 1
(July 1993)

PHOTO No. 1



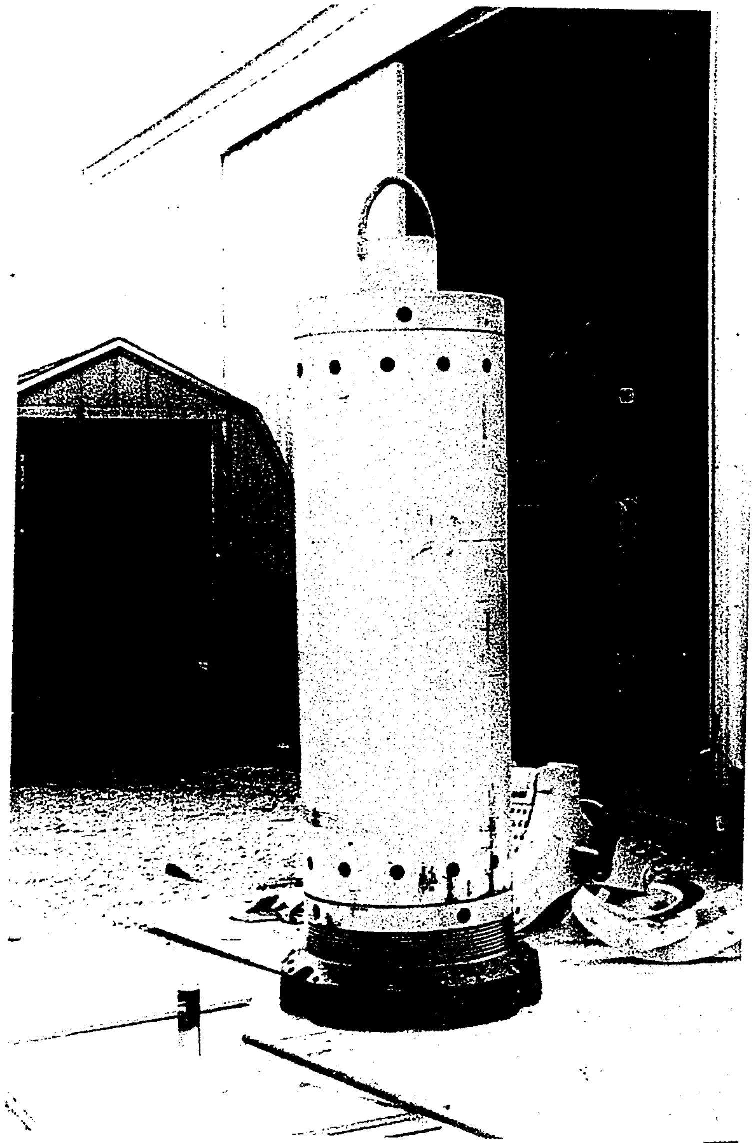
Drilling by drill rig No. 2
(July 1993)

PHOTO No. 3



Drill rigs No. 2 (left) and No. 1 (right)
and service cranes
(July 1993)

P H O T O   N o.   4



Down-the-hole hammer
INGERSOLL RAND DHD 130A
(October 1992)

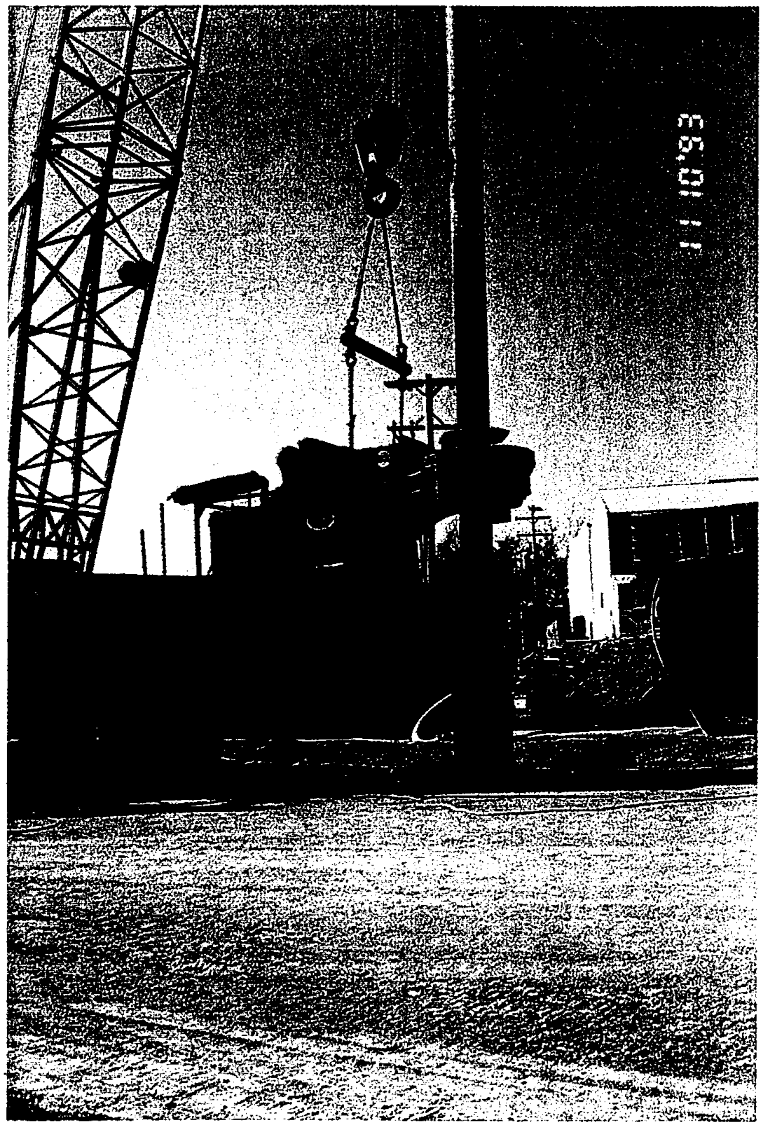
<u>PHOTO No. 5</u>

Down-the-hole hammer
SANDVIK XL24
(October 1993)

PHOTO No. 6

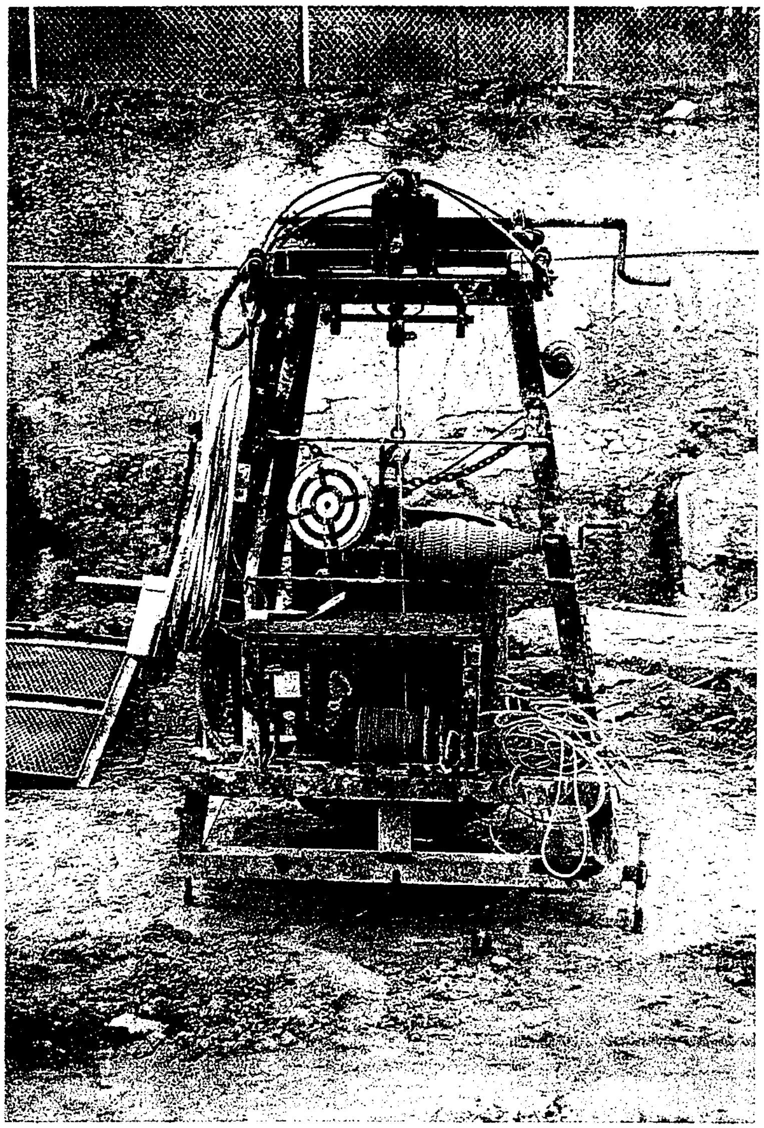
Cluster drill
INGERSOLL RAND CD24-5
(February 1993)

PHOTO No. 7



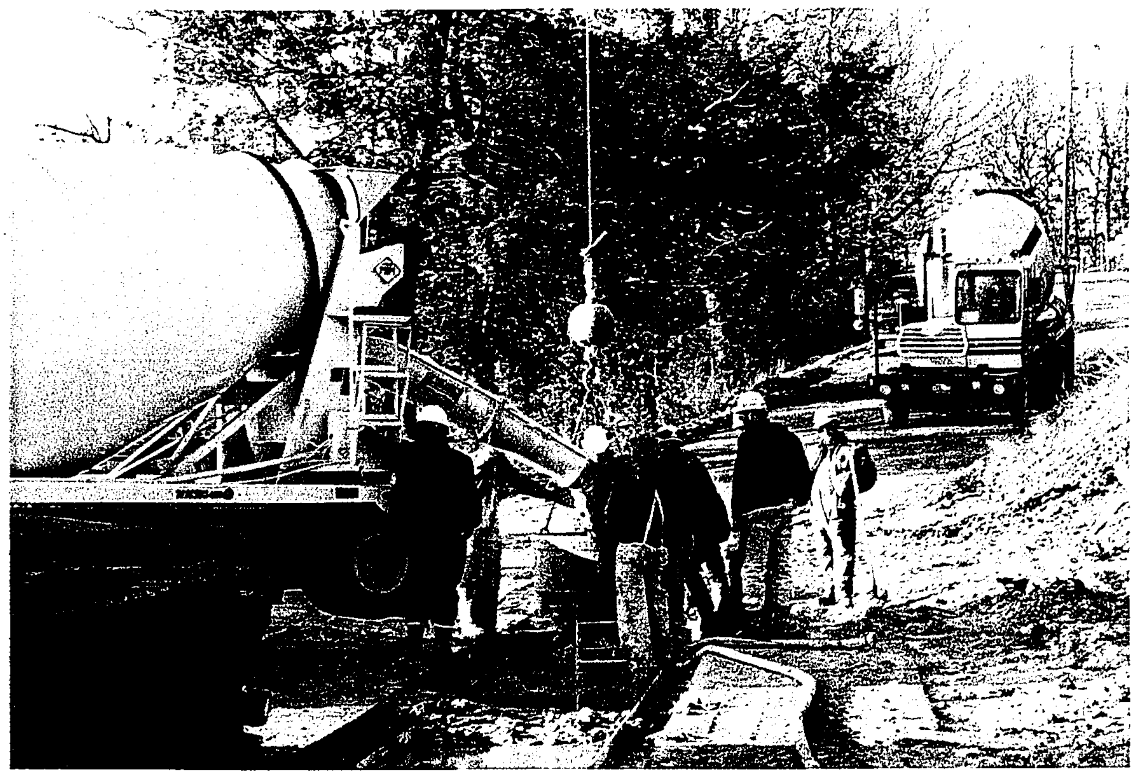
Crane LINK BELT LS338
equipped with rotary table for driving bucket
(November 1993)

PHOTO No. 8



Device for verticality measurement
(January 1994)

PHOTO No. 9

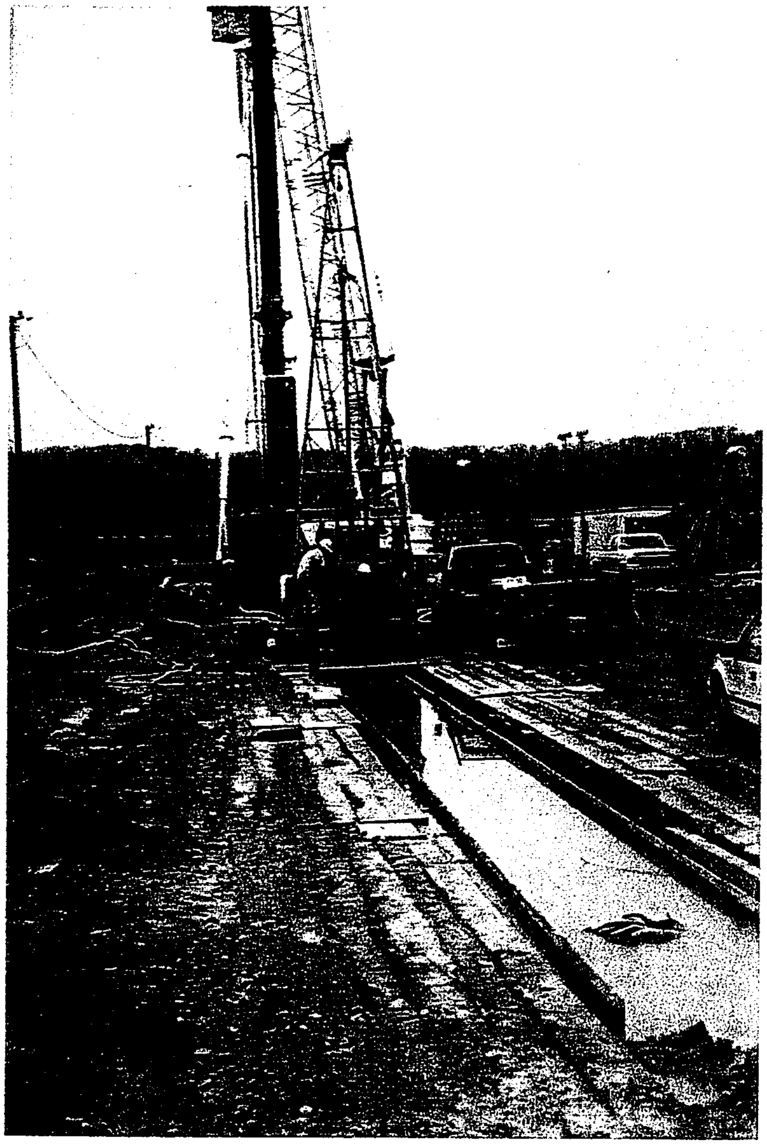


Concrete placement at the
Northern end of the cutoff wall
(November 1992)

P H O T O   N o. 10

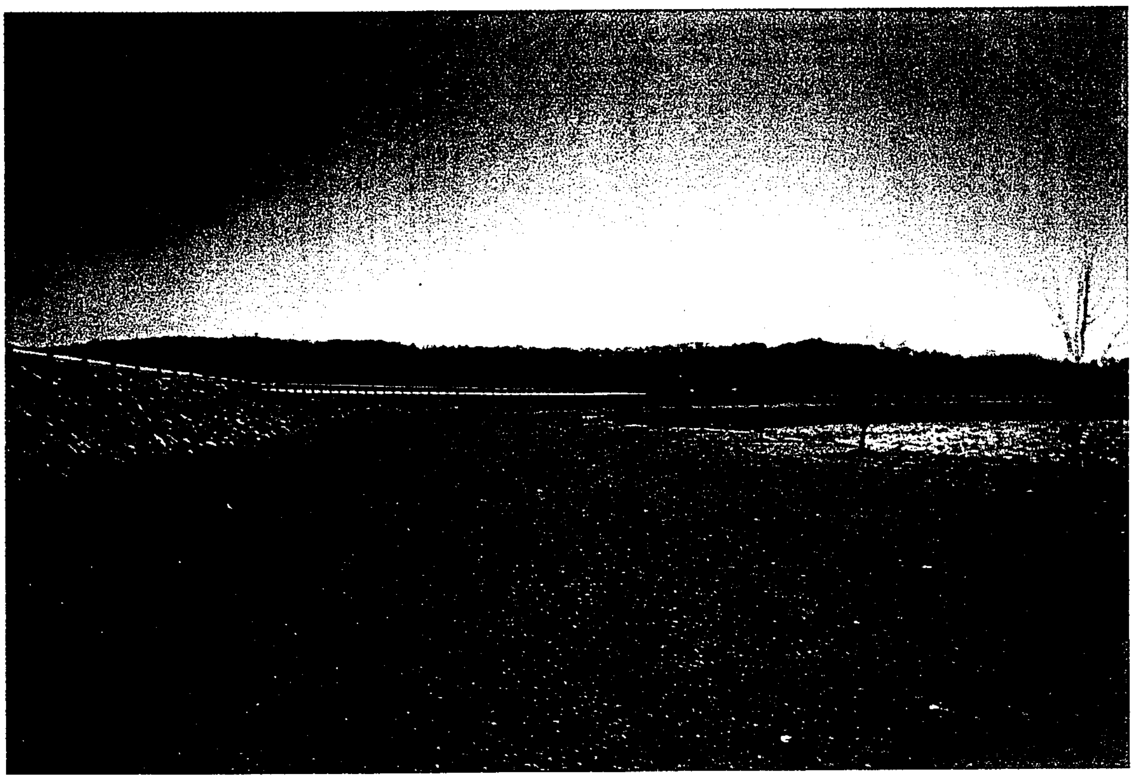
Work platform modification, at the Northern end, approaching completion (April 1993)

PHOTO No. 11



Drill rig for quality coring
(January 1993)

P H O T O N o. 12



Work platform after completion of final restoration
(looking from North to South)
(December 1994)

PHOTO No. 13

Work platform after completion of final restoration
(parking lot and Southern end)
(December 1994)

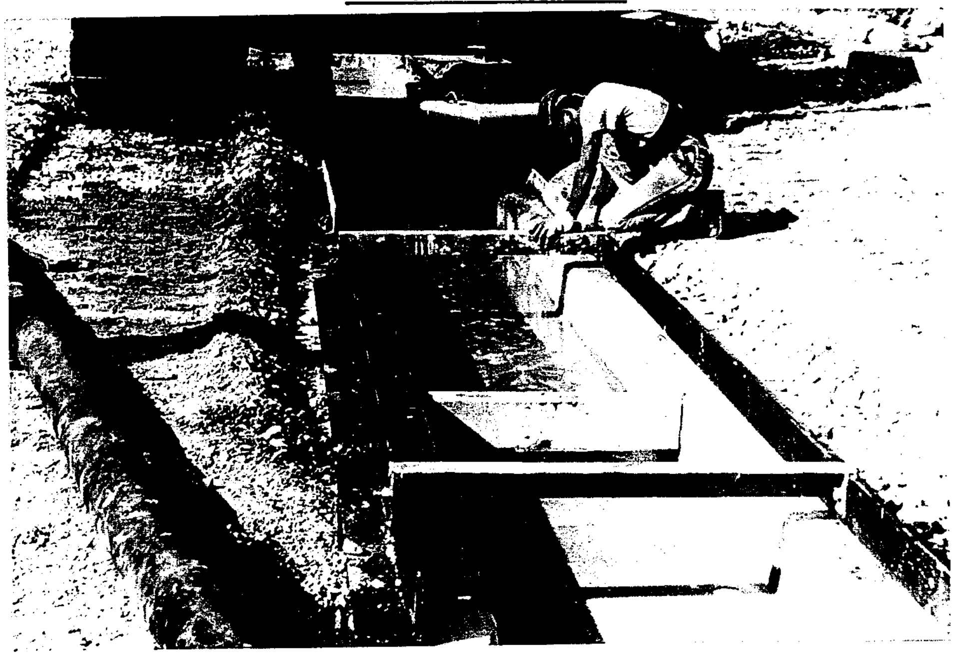
PHOTO No. 14

Seepage flow in area SA-1,
before construction of cutoff wall
(December 1993)

PHOTO No. 15

Seepage flow in area SA-1,
after construction of cutoff wall
(October 1994)

PHOTO No. 16

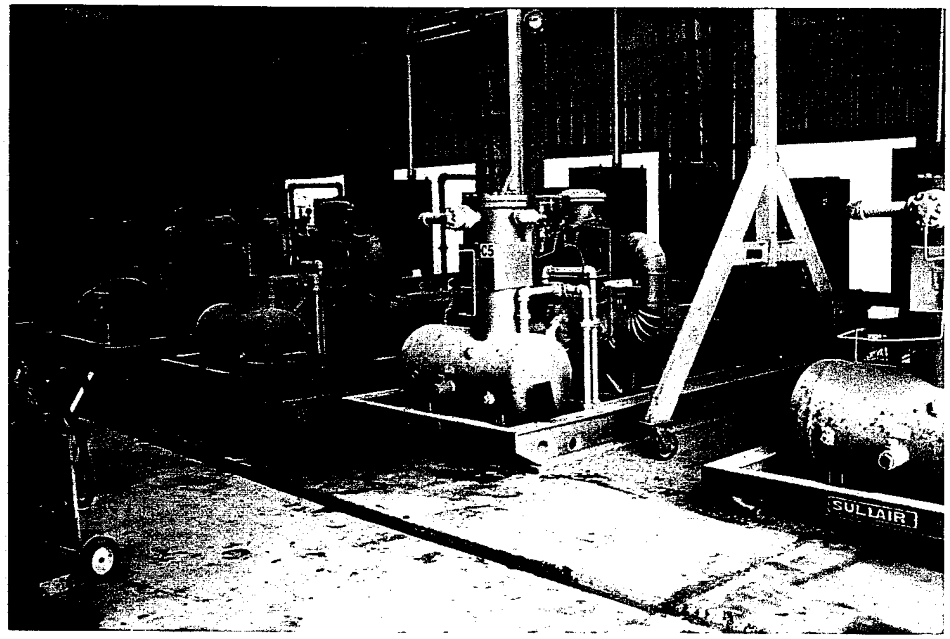


Laying guide rails (Nov. 92)

PHOTO No. 17

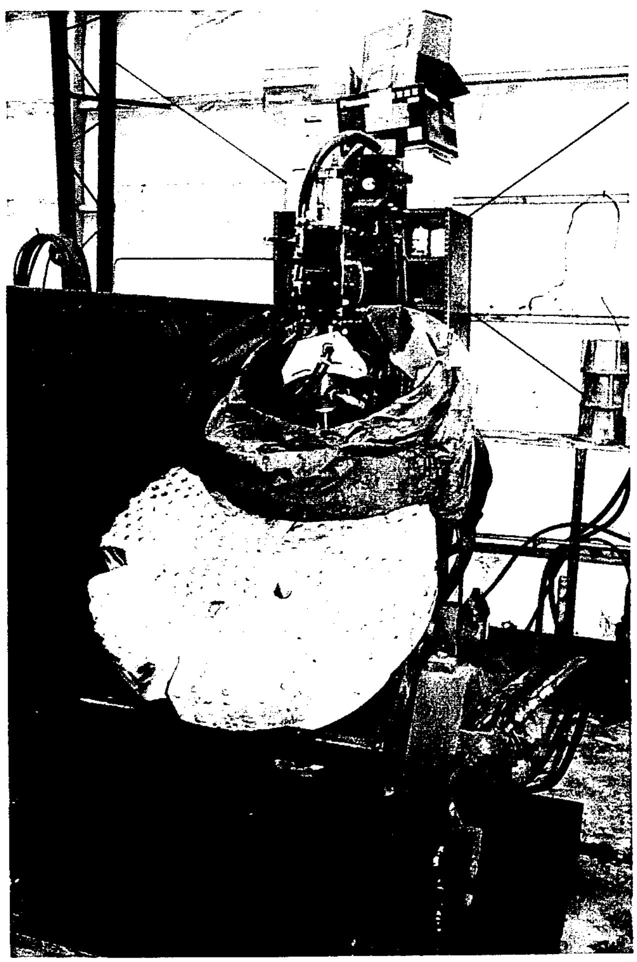
Soil stabilization guide wall being poured (Nov. 92)

PHOTO No. 18



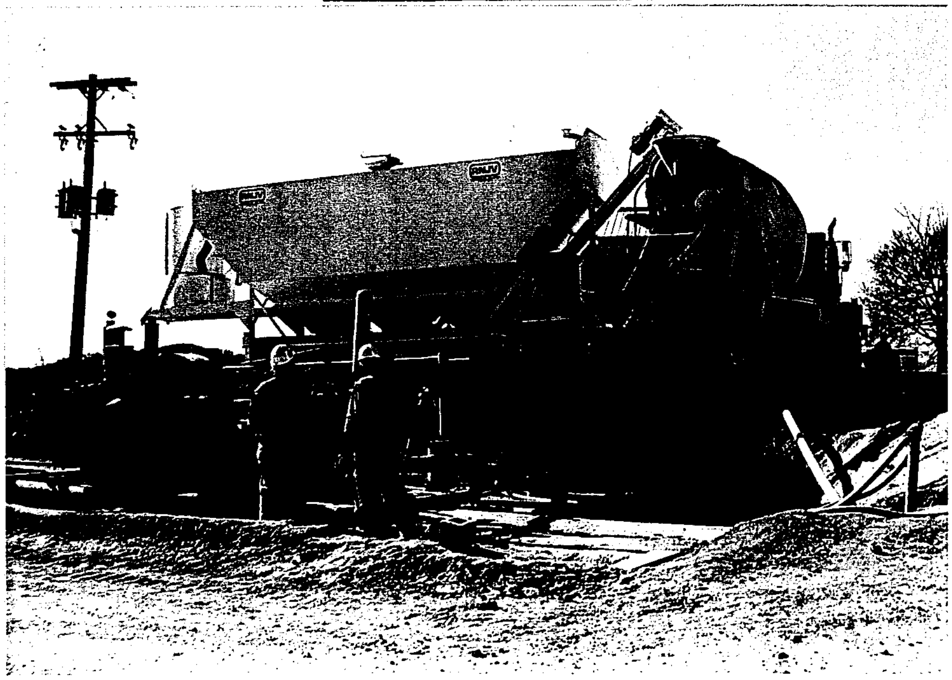
One side of RNJV'S air compressor banks (June 94)

PHOTO No. 19



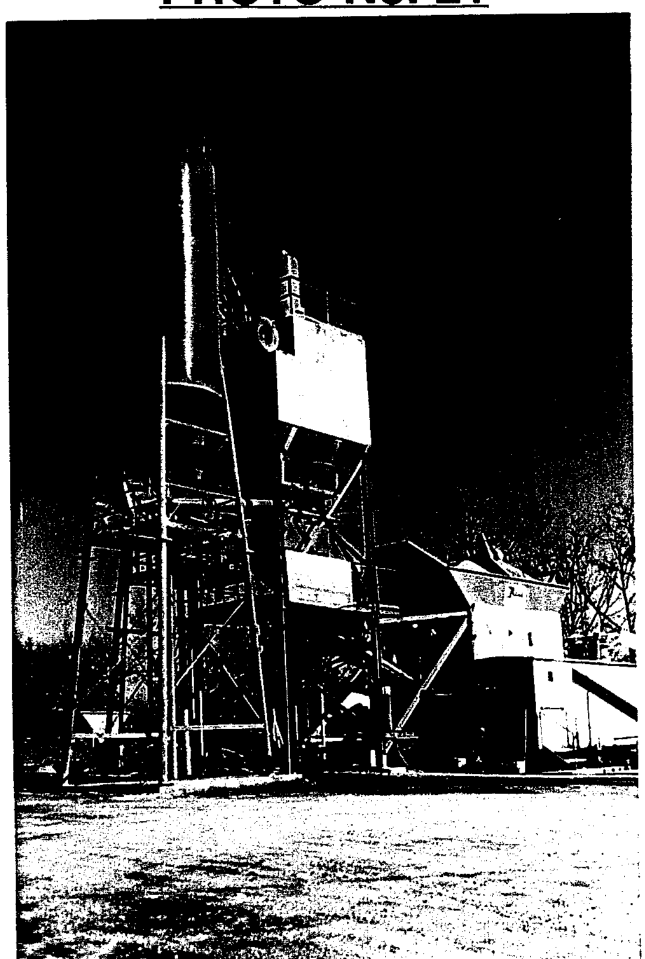
Air hammer bit being refurbished by Keystone (June 94)

PHOTO No. 20



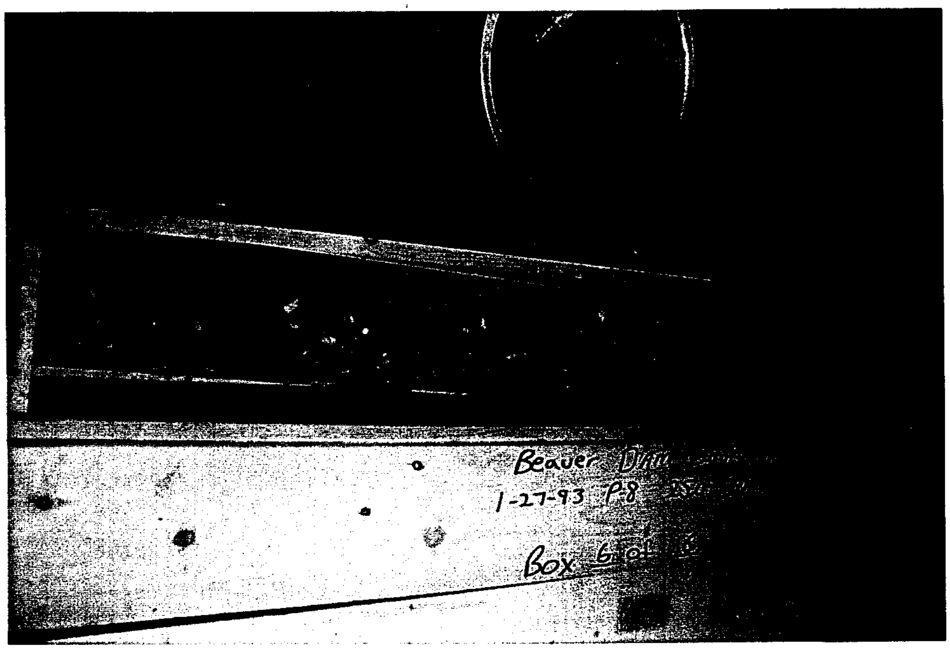
Grout Plant used in soil stabilization (March 94)

PHOTO No. 21



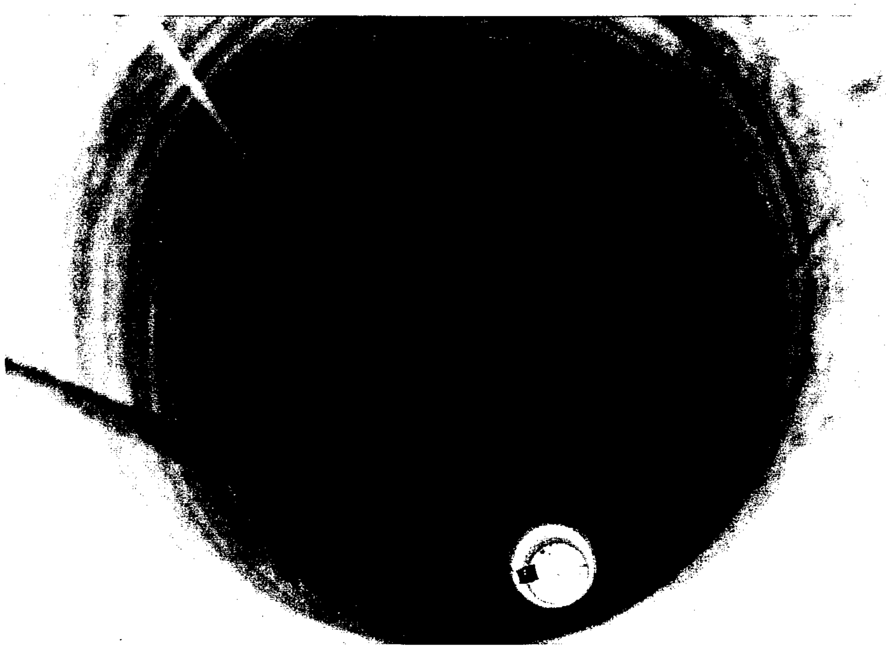
Concrete Plant (Mar. 94)

PHOTO No. 22



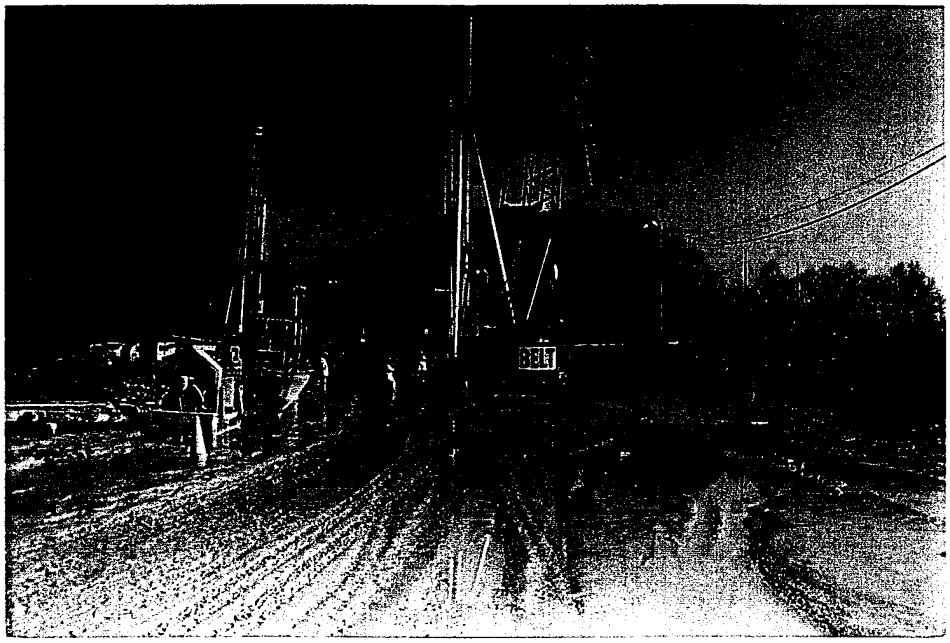
Typical concrete joint

PHOTO No. 23



Typical hole drilled with air hammer, note irregular surface provided excellent concrete bonding

PHOTO No. 24



Platform before drainage system was modified (April,93)

PHOTO No. 25

Air bubbles rising from lake approx. 200 feet from rig